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WABA DAM - PHASE II INVESTIGATION
LABORATORY TESTING REPORT

C.B.H. Cragg

Results of laboratory tests are reported and compared with in situ test measurements and original design work. These results indicate that the soft clay layer beneath the dam is underlain by more competent soil at a shallower depth than shown in the original design work. This finding could have a positive effect on settlement and stability analyses. Dynamic testing has found the soft clay to be resistant to liquefaction and post seismic event strength loss. Parameters are provided for static and dynamic stability analyses of the dam.

1.0 INTRODUCTION

The 1986 Phase I design review of Waba Dam recommended that the review be extended to Phase II/1/. The field portion of the Phase II investigation - consisting of four dilatometer soundings, two piezocone soundings and a deep sampled borehole - was performed in May and June 1987. Figure 1 shows the locations of these tests. Samples of the clay fill used in the crest and berm were obtained from the upper zones of the dilatometer holes. Samples of the foundation soils were procured from the deep borehole. This report summarizes laboratory work performed on these samples and compares laboratory values to in situ test results and original design values.

2.0 SAMPLE INVENTORY

Forty-eight samples of fill material and 35 samples of foundation material were received for testing, as listed below:

89-02616/

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Source	Number of Samples	Sample Type	Depth (ft)	Soil Type
Dilatometer 1	8 2	Spoon 2" Shelby	5 - 37	Fill
Dilatometer 2	7 3	Spoon 2" Shelby	5 - 38	Fill
Dilatometer 3	9 2	Spoon 2" Shelby	5 - 41	Fill
Dilatometer 4	7 1	Spoon 2" Shelby	5 - 40	Fill
Borehole 2	6 3	Spoon 2" Shelby	5 - 41	Fill
Borehole 2	13	5" Osterberg	51 - 112	Foundation clay
Foundation				
Borehole 2	12	3" Osterberg	41 - 48	Foundation clay
Borehole 2	3	2" Shelby	231 - 247	Foundation clay
Borehole 2	7	Spoon	224 - 226	Foundation clay
			250 - 287	Foundation till

As each 5" Osterberg provided enough material for at least 15 test specimens, there was no shortage of upper foundation clay or surface fill for testing. Comparatively fewer samples of lower foundation soils were obtained, necessitating multiple usage of certain samples to provide the desired information.

3.0 STRATIGRAPHY

To simplify this presentation of laboratory test results the soils have been divided into 5 strata:

1. clay fill (crest or berm)	0 - 40'
2. upper clay	40 - 145'
3. lower clay	145 - 250'
4. sand	250 - 253'
5. till	253 - 287'

as shown on Figures 2 and 3. Figures 4 and 5 show that NRC and OHRD piezocone tests support these fill/upper clay/lower clay divisions.

The NRC piezocone (Figure 4) indicates the presence of a harder layer at 218 feet which is not supported by the sampling and laboratory test program. Results of the OHRD piezocone and dilatometer tests are not useful in establishing layer boundaries as these soundings did not penetrate into the lower clay stratum.

Within the upper clay, three sub layers are suggested by the patterns of water content, bulk density and shear strength tests. The piezocone and dilatometer tests do not show this pattern. Designers are encouraged to review this interpretation of stratigraphy before employing it in design review analyses.

4.0 LAYER PROPERTIES

The layer properties are summarized by category of test - index tests, undrained shear strength, effective shear strength, compressibility and dynamic properties - in the following sections.

4.1 Index Tests

As may be seen from Figure 3, there is a certain amount of scatter in the properties of the clay fill. On average the fill has a water content of 40%, a bulk density of 118.6 lb/ft^3 (1900 kg/m^3) and an undrained shear strength of 2100 psf (100 kPa). Water content increases and bulk density decreases from 35 to 40 feet may be reason to review the fill to foundation soil boundary.

From Figure 2, it may be seen that the till has a water content of about 10%. One bulk density test yielded a value of 148 lb/ft^3 (2371 kg/m^3) for the till. Insufficient material was available for further density or shear strength tests on the till.

One sand sample yielded a water content of about 15% (Figure 2).

The lower clay has a plastic limit of 20%, a liquid limit of about 40% and a natural water content near the liquid limit (Figure 2). Bulk density of the lower clay is about 112 lb/ft^3 (1800 kg/m^3) and undrained shear strength increases with depth from 3100 psf (150 kPa) to 4200 psf (200 kPa) (Figure 2).

The upper clay has water contents ranging between 70% and 90% to a depth of 100 feet, then decreasing to 50% at 140 feet below grade. Bulk density generally ranges from 94 lb/ft^3 (1500 kg/m^3) to 100 lb/ft^3 (1600 kg/m^3) but values on the order of 106 lb/ft^3 (1700 kg/m^3) were measured at the bottom of this layer. Shear strength for the upper clay averages 1040 psf (50 kPa). Figure 2 indicates that there is a pattern to the water content, bulk density and shear strength measurements in the upper clay that may justify subdividing this stratum into two or three layers. Specific gravity for the clay ranges from 2.74 to 2.78 (Table I).

4.2 Undrained Shear Strength

In contrast to the three layer pattern of shear strength in the Upper Clay revealed by laboratory tests (Figure 2), the in situ tests show a uniform increase in shear strength with depth (Figures 4, 5 and 6) from 630 psf (30 kPa) to 1460 psf (70 kPa). Shear strength values from the in situ tests are very consistent in this Upper Clay. The NRC piezocone (Figure 4) agrees closely with the OHRD piezocone (Figure 5). The dilatometer tests show similar results from several locations across the dam (Figure 6) and agree closely with the piezocone results (Figure 7). Laboratory test values follow the trend of the in situ tests but display more scatter due to the apparent layering (Figure 8). Figure 9 shows the current laboratory tests agree reasonably well with shallow dilatometer and vane tests by Bechai/2/ at the dyke toe in 1985.

In the Lower Clay, both the NRC piezocone and laboratory tests show a pattern of increasing shear strength with depth (Figure 10). However, the piezocone results in this layer are substantially lower than the laboratory test values. This difference may arise from selection of the Nk factor used to interpret shear strength from the piezocone results. The piezocone values increase from 1880 psf to 3130 psf (90 to 150 kPa) between the depths of 141 to 246 feet (43 to 75 m). Laboratory test values increase from 2920 to 4180 psf (140 to 200 kPa) over the same depth interval.

The current laboratory test values appear to be lower than values measured by Acres in their 1972 test program in the Upper Clay but similar to Acres' values in the Lower Clay (Figure 11). However, Figure 11 shows that current shear strength measurements are equal to or greater than the shear strength values used in Acres' 1976 stability analyses.

4.3 Effective Shear Strength Parameters

Four sets of static triaxial tests yielded the following results:

Set	Soil	Depth (ft)	c' (kPa)	ø'	Remarks
A	Clay Fill	15 - 20	21.5	30.5°	from Dil. 2 (crest)
B	Upper Clay	67	8	22.5°	sample with weak layer, still consolidating when sheared
C	Upper Clay	67	30	27.5°	staged consolidation
D	Lower Clay	170	22	30°	multi-stage test

Consolidation curves, stress-strain plots and other details of these tests are contained in Appendix A. Shear strength parameters for the fill and the lower clay are nearly identical. The upper clay is slightly weaker (Set C) to much weaker (Set B). Three factors contribute to the lower strength of Set B: (1) presence of a 10 mm thick weaker layer in the samples, (2) high

volume change suggesting destruction of the soil structure during consolidation, and (3) consolidation was incomplete at the time the samples were sheared to failure (see consolidation curves in Appendix A). These lower strength parameters may be appropriate for the zone beneath the dam that was deliberately stressed beyond its preconsolidation pressure in the original design. Whether the weak seam represents only a local feature or recurring conditions through the soil mass is not known.

4.4 Compressibility

Consolidation tests results show the upper clay has preconsolidation pressures increasing from 2300 psf to 4800 psf with depth (see Appendix B). Preconsolidation pressures in the lower clay ranged from 6000 psf to 16,000 psf. Figure 12 shows these samples are over consolidated with respect to original grade and underconsolidated with respect to the current berm fill level. Coefficient of consolidation values range from 0.004 to 0.0285 $inch^2$/minute in the 1000 to 8000 psf stress range for the upper clay and 0.0499 to 0.1991 $inch^2$/minute for the lower clay (Appendix B).

Interpretation of dilatometer test results also yields a measure of OCR (over consolidation ratio) and preconsolidation pressure. The OCR results in the foundation clays were generally less than 1.0, indicating the soil is under consolidated under the present overburden pressure. Estimates of preconsolidation pressure from the dilatometer were much higher than the laboratory measured values (Figure 13). This difference is likely caused by an incorrect estimate of the effective overburden pressure by the dilatometer analysis program.

4.5 Dynamic Soil Properties

Resonant column tests and dynamic triaxial tests were used to evaluate soil properties pertinent to seismic and post-seismic analyses. Results of thirteen resonant column tests, six on the upper clay and seven on the lower clay, are detailed in Appendix C and summarized on Figures 14, 15 and 16. It may be observed that shear modulus increases with depth from 115 tsf (11 MPa) to 1220 tsf (117 MPa) (Figure 14). With the exception of the lower 40 foot portion of the upper clay, the increase of shear modulus with depth is generally linear. Previous tests/3/ in the upper clay yielded higher shear modulus results than two samples tested from this depth region in the current program. Changes in the pattern of shear modulus versus depth occur at the same depths as changes in index properties in Figure 2.

The ratio of shear modulus to shear strength appears relatively constant with depth and has an average value of approximately 500. This ratio value agrees well with previous work at Long Sault/4/ and Cornwall/5/ where ratios of 600 and 500 respectively were obtained.

Figure 15 shows that the shear modulus for the upper clay decreased 53% with increasing strain. In the same strain range, damping increased from 2.3% to 5.2%. For the lower clay, increased strain reduced shear modulus values by 60% and increased damping from 1% to 5% (Figure 16). Figure 15 shows that the theoretical curve matches the modulus decrease with strain quite well in

the upper clay but Figure 16 shows this match is not good in the lower clay. The pattern of modulus and damping change with strain of the upper and lower clay is quite similar, with the exception of initially lower damping values in the lower clay (Figure 17). Figure 18 compares the results of current tests in the upper clay to work done in 1985/3/. Damping behaviour is virtually identical. Current work shows a greater decrease in modulus with strain than measured in the 1985 work (Figure 18).

4.6 Resistance to Dynamic Loading

Two types of tests were used to evaluate the effects of dynamic loading on Waba Dam foundation clays. Dynamic triaxial tests were performed to assess the susceptibility of these marine clays to liquefaction. Static (quick-undrained) triaxial tests were performed on clay samples that had been subjected to limited dynamic loading to determine "post earthquake" strength reduction.

In the dynamic triaxial tests, a series of samples were tested at various stress ratios (dynamic shear stress/confining pressure) to obtain a relationship between stress ratio and number of cycles to "failure". Repeat series were performed at different confining pressures to evaluate the liquefaction resistance of foundation clay at different depths below the dam. Samples were selected for these series from depths of 63, 92 and 110 feet to represent each of the sublayers of the upper clay and from depths of 190 to 200 feet within the lower clay. The samples at 63 feet and 110 feet were the weakest materials encountered in the top and bottom sublayers of the upper clay. Analyses by Wai/6/ indicate that the maximum stress ratio to be expected below Waba Dam is about 0.26. This value is nearly identical to the expected maximum stress ratio calculated below Long Sault Dam/7/. In the dynamic triaxial tests, the clay samples were subjected to stress ratios of 0.28 to 0.42 (approximately equivalent to Magnitude 6 to 8 earthquakes). Clay from the 92 foot depth did not fail under 300 plus cycles of these stresses while the samples from 63 and 110 feet did liquefy. Figures 19 and 20 show that the Waba Dam clay exhibits dynamic load resistance similar to or better than the Cornwall (Long Sault) clay. Figure 20 indicates that the cyclic strength of marine clays from Waba Dam and Cornwall is greater than that of weak materials known to have performed poorly under actual earthquake conditions (eg Lower San Fernando Dam) and similar to materials that have performed well during actual earthquakes. Details of these tests are shown in Appendix D.

To evaluate the "post earthquake" loss of strength of these clays, samples were subjected to 10, 25 or 100 cycles of 0.28 stress ratio then immediately loaded statically to failure. These test results show virtually no strength

loss compared with reference shear strength values (Appendix D). Limited testing of Cornwall clay in thi s manner also showed no strength loss/7/.

4.7 Pore Water Chemistry

Salt content tests were performed on the pore water extracted from ten Borehole 2 clay samples. Three samples were from the clay fill layer, four were from the upper clay stratum and three were from the lower clay stratum. Figure 21 shows the test results in two forms. One form is compared with results reported by Acres in 1972/8/.

The test method used for current work is probably different than that used in 1972, which may account for some of the differences in the results. In current tests, pore water is squeezed from the clay samples in a press, passed through a 45 μm filter to remove sediment and then tested for total conductivity. Using the conversion methods shown in Appendix E, pore water conductivity results are translated into equivalent sodium chloride (NaCl) salt content. This salt content is expressed in two ways: (1) grams NaCl per litre of pore water (g/ℓ), (2) grams NaCl per 100 grams of dry soil (%). In 1972, the common practice for determining salt content involved drying the soil and subsequent soluble salt extraction using distilled water/9/. As the Acres (1972) results were reported as % dry soil, it is believed the drying method was employed for their work. The drying method yields a higher estimate of pore water salinity than the current squeezing method/9/.

From Figure 21, it may be seen that salinity is low in the clay fill and high in the native clay, with values decreasing towards the underlying sand layer. In the natural clay, salinity generally ranges from 15 to 24 g/ℓ. Near the lower sand a value of 7 g/ℓ was obtained. The salinity of sea water is about 26 g/ℓ. Salinity affects the Atterberg limits, sensitivity and compressibility of marine clay/9/. Between 26 g/ℓ and about 10 g/ℓ, the liquid limit is almost constant. Below 10 g/ℓ, the liquid limit decreases until below 2 g/ℓ, conditions are reached at which quick clay can exist. At low salinities, marine clay exhibits very high sensitivity and an increased rate of secondary consolidation.

The current salinity results are similar to values obtained by Acres in the clay crust and lower clay layers. Most of the Acres data is inexplicably much lower than current salinity values in the upper clay layer, though peak values are quite similar.

4.8 Swelling Properties of Clay Fill

Due to the unusual behaviour of some deep settlement points, a few swelling tests were performed on the clay fill. Using standard oedometer equipment, four fill samples were loaded to 2800 psf (to represent the overburden pressure at the base of the dam) and then saturated and allowed to swell. As the results shown in Appendix F demonstrate, the samples did not exhibit any tendency to swell during the 8 day test period. In fact, a settlement of about 0.05% was observed during the "swelling" period.

5.0 SUMMARY

Results of the laboratory testing associated with the Dam Safety Program Phase II investigation of Waba Dam have been reported. These results generally show a good correlation with recent in situ tests. Interpretation of these results yields a stratigraphy of two major clay strata underlain by a thinner competent till stratum. The upper weaker clay stratum can potentially be divided into three sub units. This stratigraphy concept is different from the one used in the original design of Waba Dam. Shear strengths measured in the current program were generally lower than those obtained by the initial design investigation but equal to or higher than values employed in original design calculations. Selection of higher than original design shear strengths can be considered for use in the design review. Consolidation tests indicated the deep clay foundation has not yet fully consolidated under the load caused by construction of Waba Dam. Dynamic tests indicate that the marine clay will perform well during seismic events and that no significant strength loss will occur after a seismic event of expected magnitude. A complete summary of soil properties is provided for use in design review analyses.

ACKNOWLEDGMENTS

Field sampling, dilatometer tests and piezocone tests were performed jointly by Mr. M. Grills and Mr. A Coutie (GHED). Mr. P. Toth and staff from NRC, particularly Dr. K.T. Law, were instrumental in accomplishing the piezocone tests. Mr. P. Adams performed most of the dynamic and static triaxial tests. Resonant column work was performed primarily by Mr. J. Jones. Much credit is due to Mr. A. Vorauer for computer analysis and presentation of the test results. Marshall Silver made a timely and useful review of the dynamic testing work.

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3. C.B.H. Cragg. "Waba Dam Dynamic Soil Properties". Research Division Report No 85-129-K, 1985.

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8. Acres 1972 Drawings.

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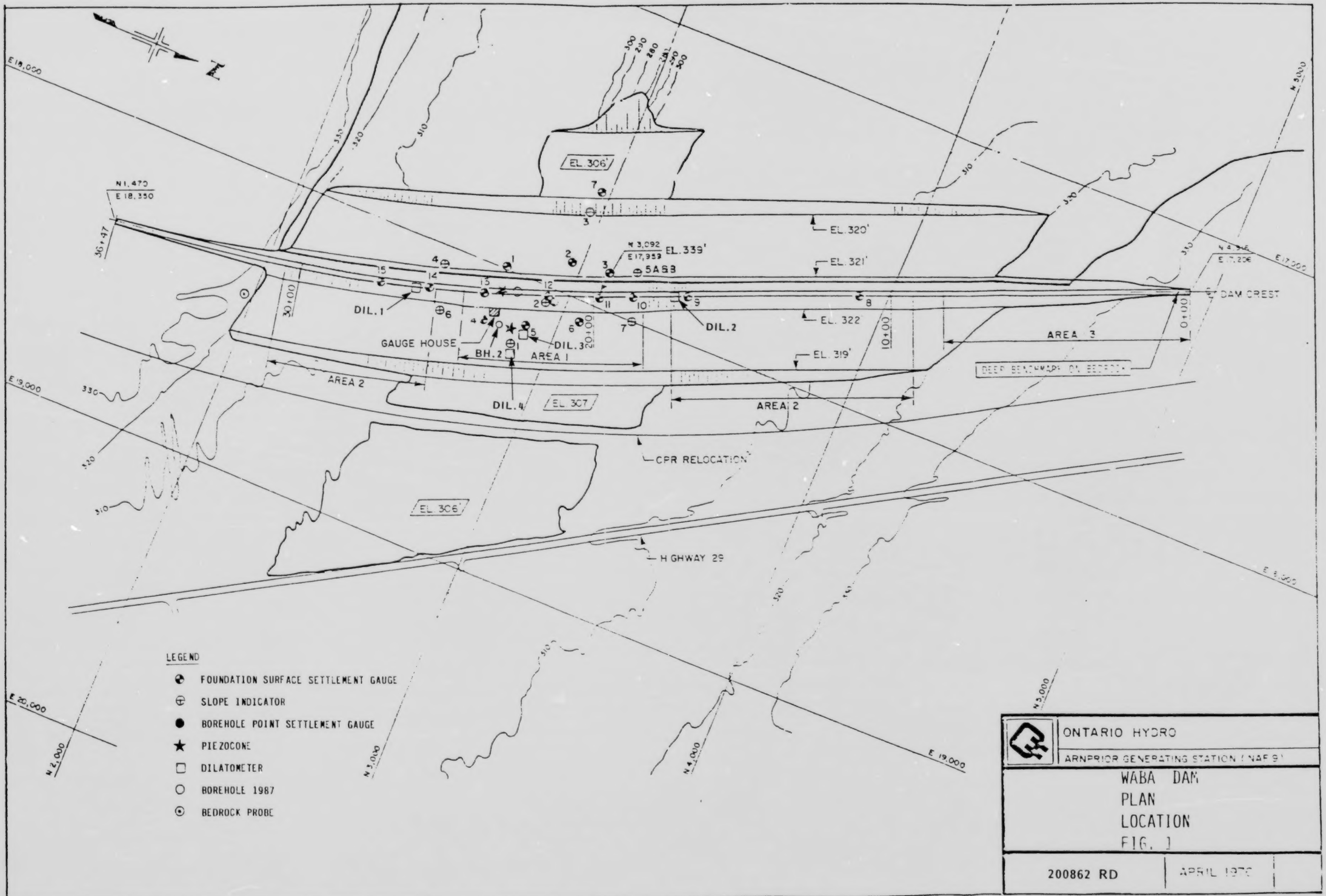
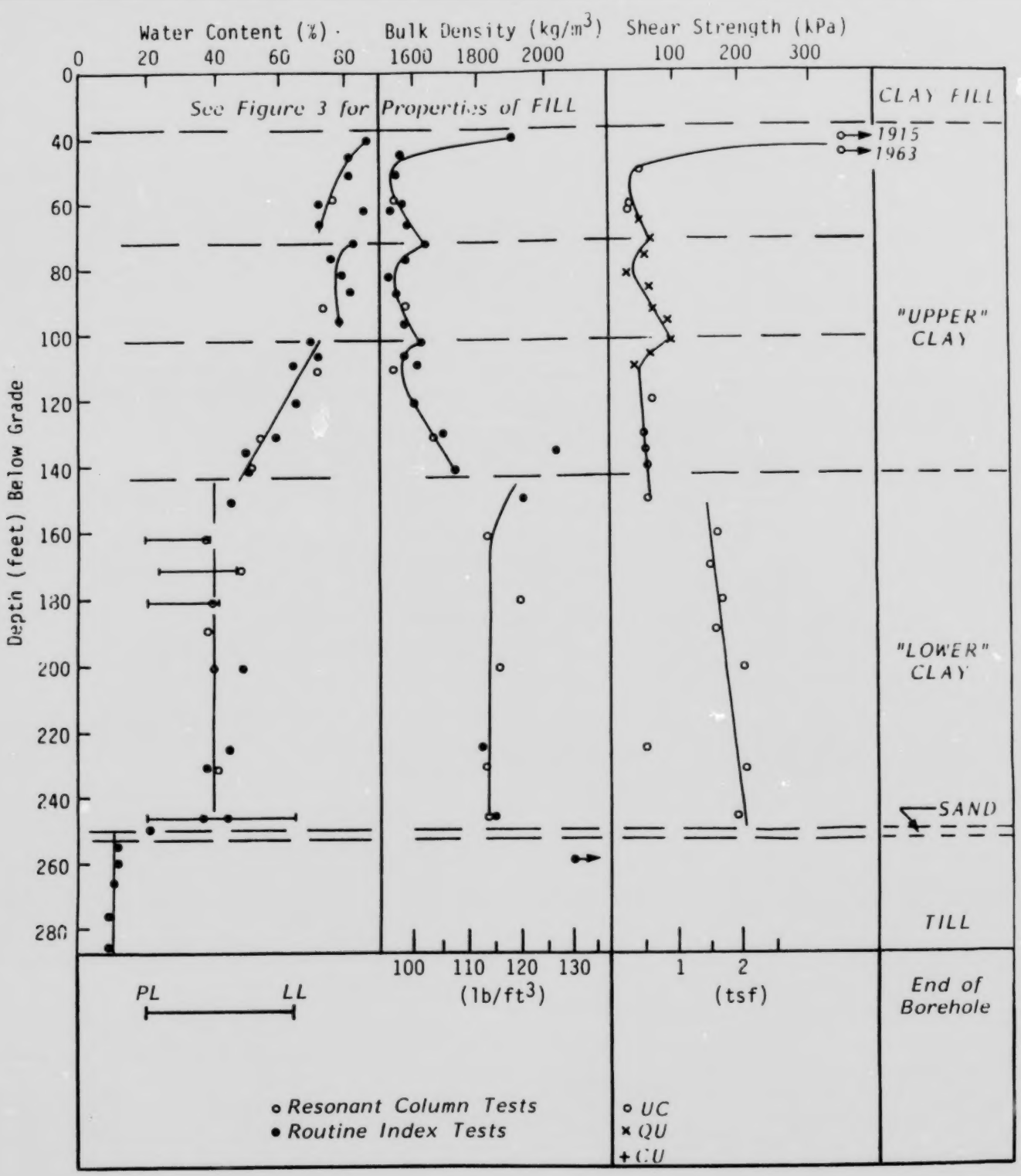
TABLE I

Waba Dam Specific Gravity Tests

Sample No	Depth (ft)	Specific Gravity
4A	59	2.78
4B	59	2.76
11A	92	2.77
11B	92	2.74
16A	121	2.74
16B	121	2.78
18A	136	2.78
18B	136	2.75
	AVERAGE	2.76

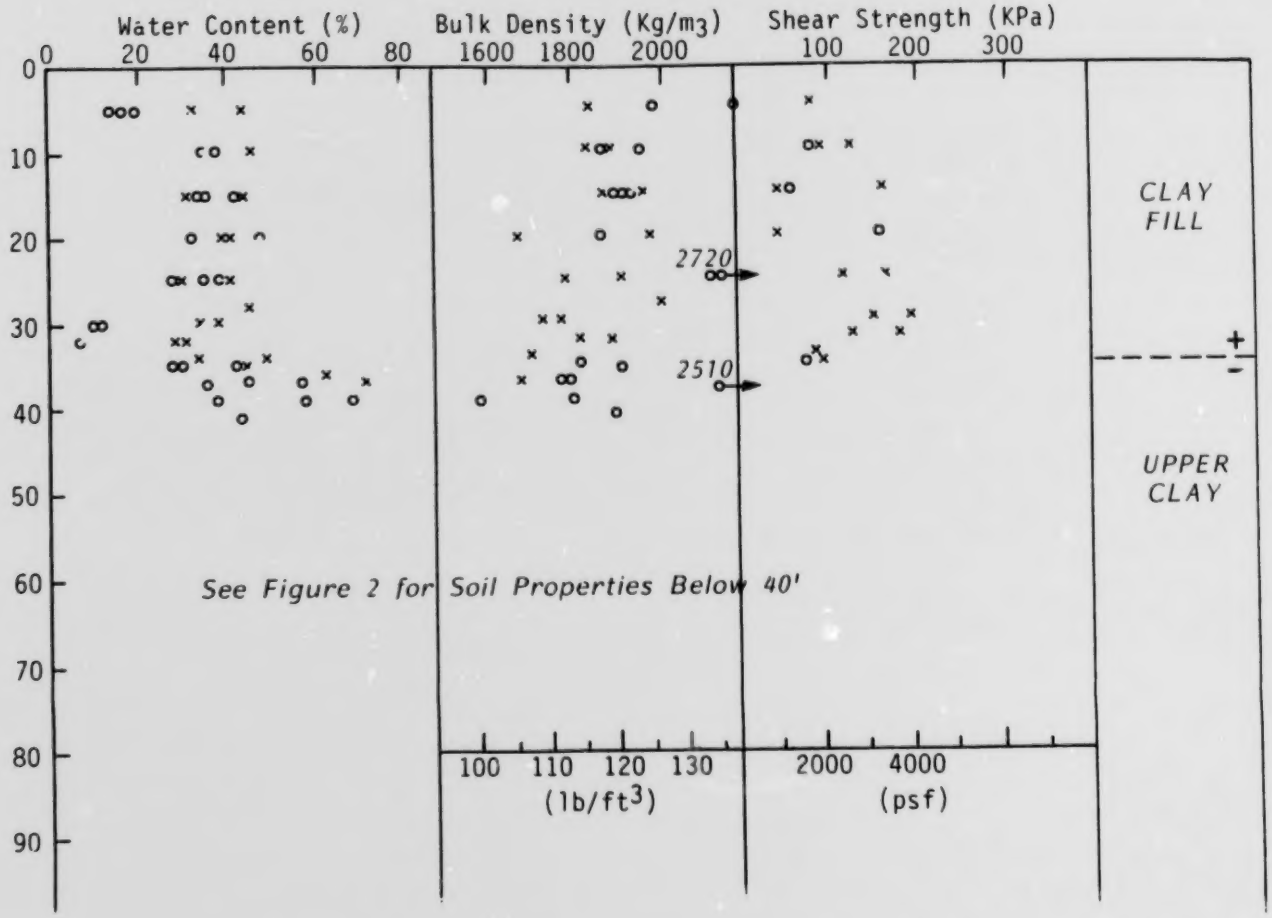

EL. 306'
EL. 320'
EL. 321'
EL. 339'
EL. 322
EL. 319'
EL. 307
EL. 306'
DIL. 1
DIL. 2
DIL. 3
DIL. 4
GAUGE HOUSE
BH. 2
AREA 1
AREA 2
AREA 2
AREA 3
DAM CREST
DEEP BENCHMARK ON BEDROCK
CPR RELOCATION
HIGHWAY 29
N1,470
E18,350
N3,092
E17,953
N3,515
E17,206
E17,000
E19,000
E18,000
E20,000
N2,000
N3,000
N4,000
N5,000
36+47
30+00
20+00
10+00
0+00
LEGEND
FOUNDATION SURFACE SETTLEMENT GAUGE
SLOPE INDICATOR
BOREHOLE POINT SETTLEMENT GAUGE
PIEZOCONE
DILATOMETER
BOREHOLE 1987
BEDROCK PROBE
ONTARIO HYDRO
ARNPRIOR GENERATING STATION (NAES)
WABA DAM
PLAN
LOCATION
FIG. 1
200862 RD
APRIL 1970



* All Samples from BH.2 (Elevation 318.5')

FIGURE 2

WABA DAM - PHASE II
SOIL INVESTIGATION



× *Crest (Dil.1; Dil.2)*

o *Berm (Dil.3; Dil.4; BH.2)*

FIGURE 3

WABA DAM - PHASE II SOIL INVESTIGATION

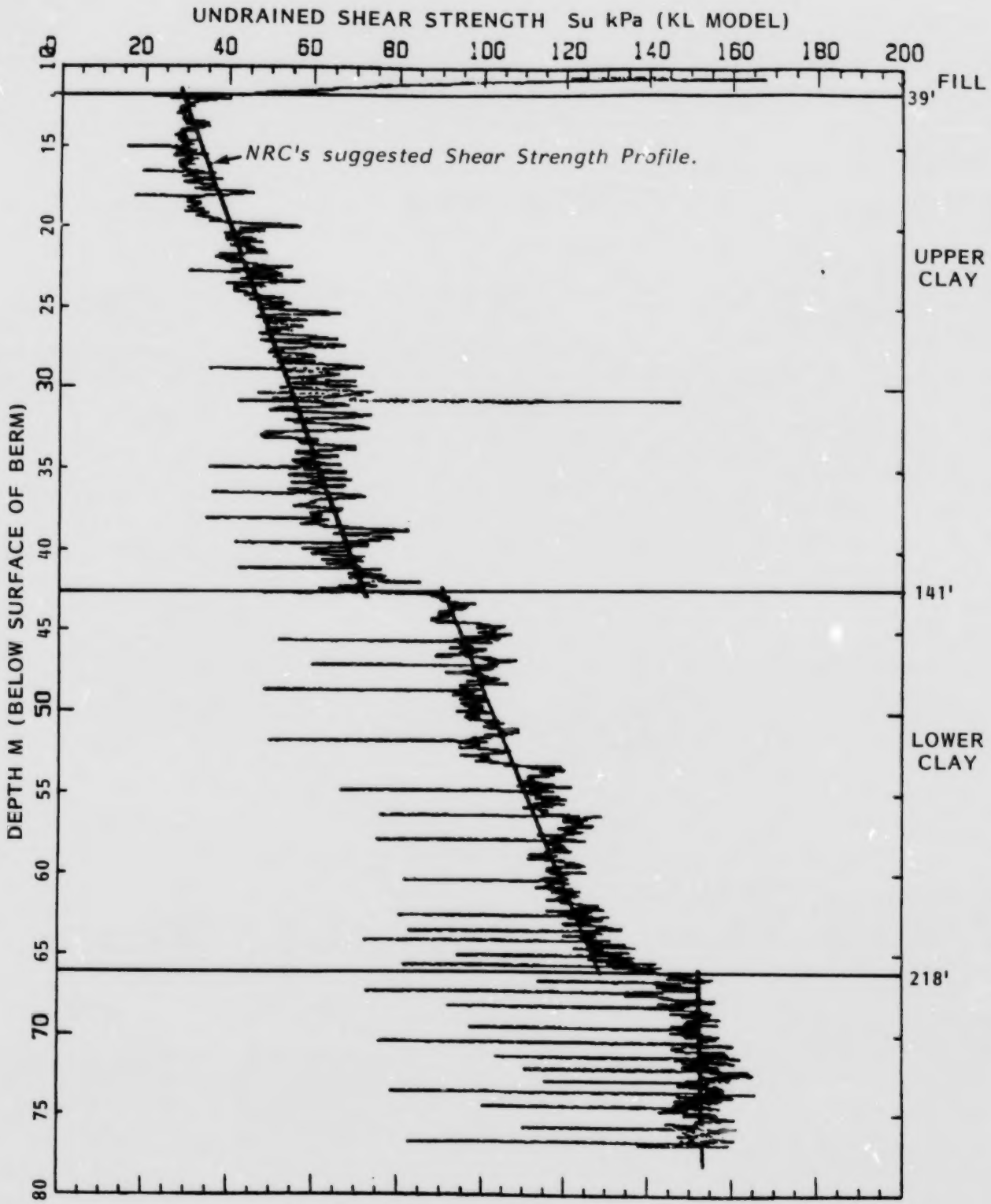


FIGURE 4

NRC PIEZOCONE TEST - ON BERM (1987)

176177 RD

87-329

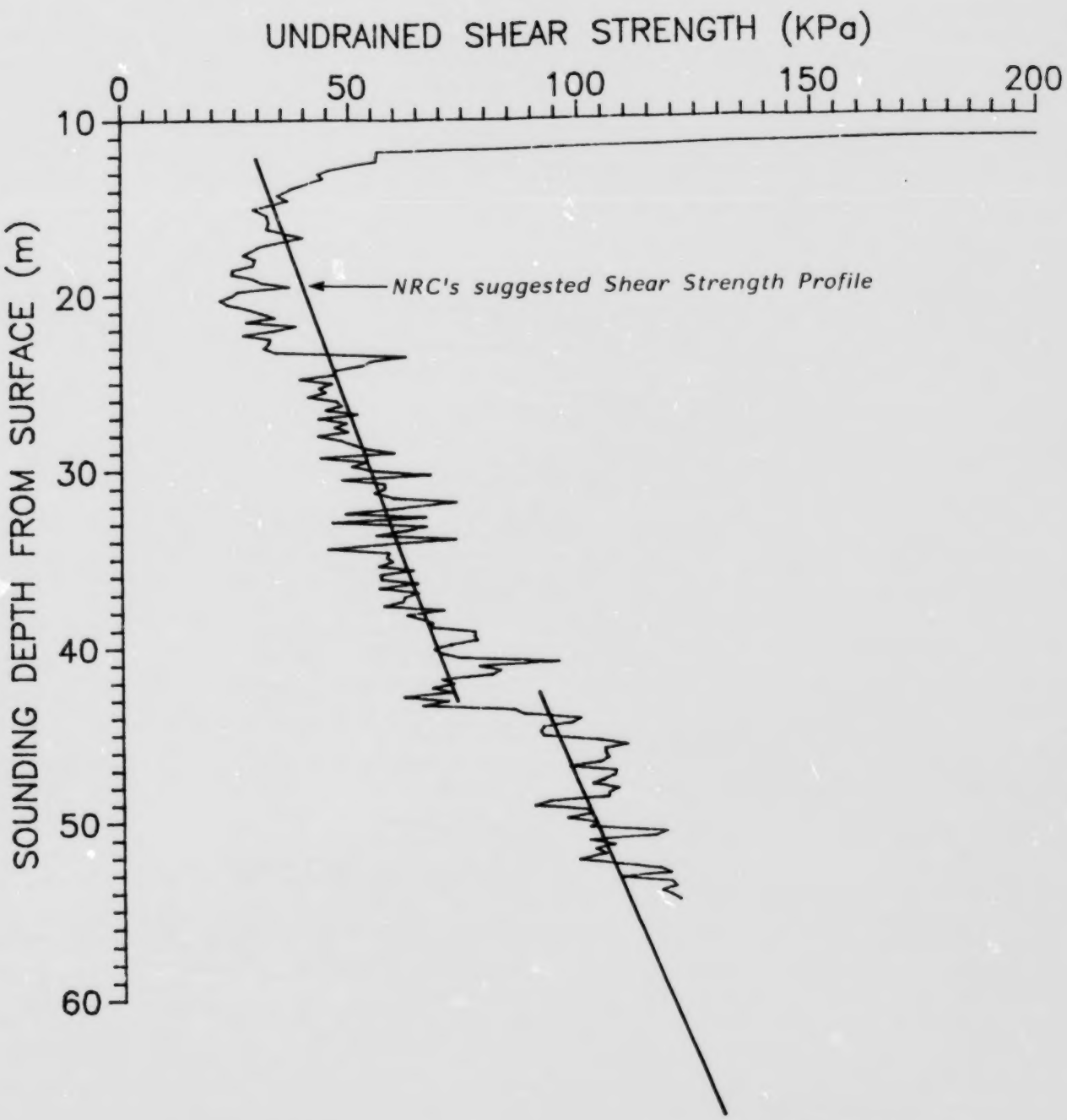


FIGURE 5

OHRD PIEZOCONE TEST ON BERM

FIGURE 6

UNDRAINED SHEAR STRENGTH AT WABA DAM
MEASURED WITH THE DILATOMETER

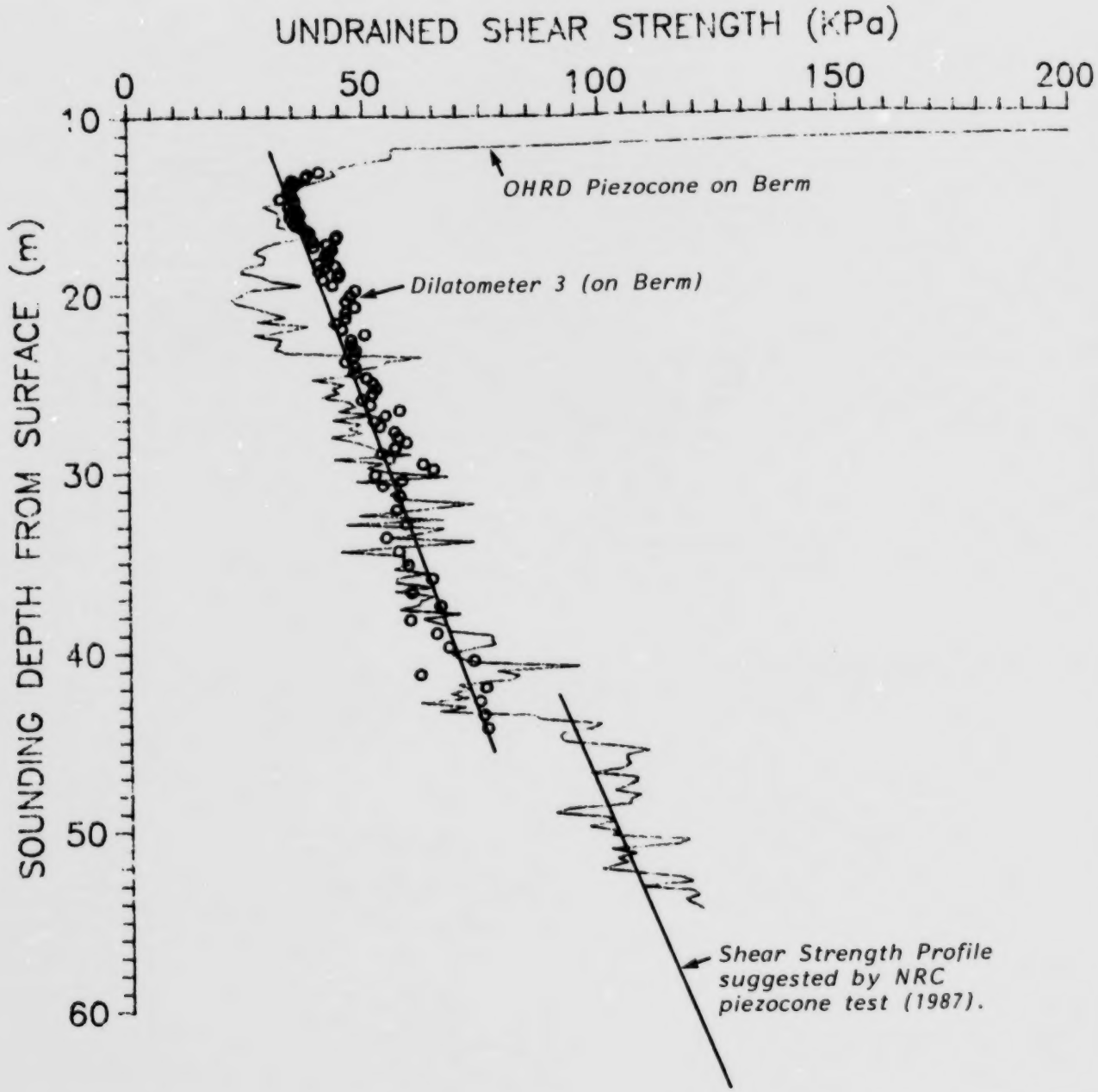


FIGURE 7

COMPARISON OF IN SITU TESTS AT WABA DAM

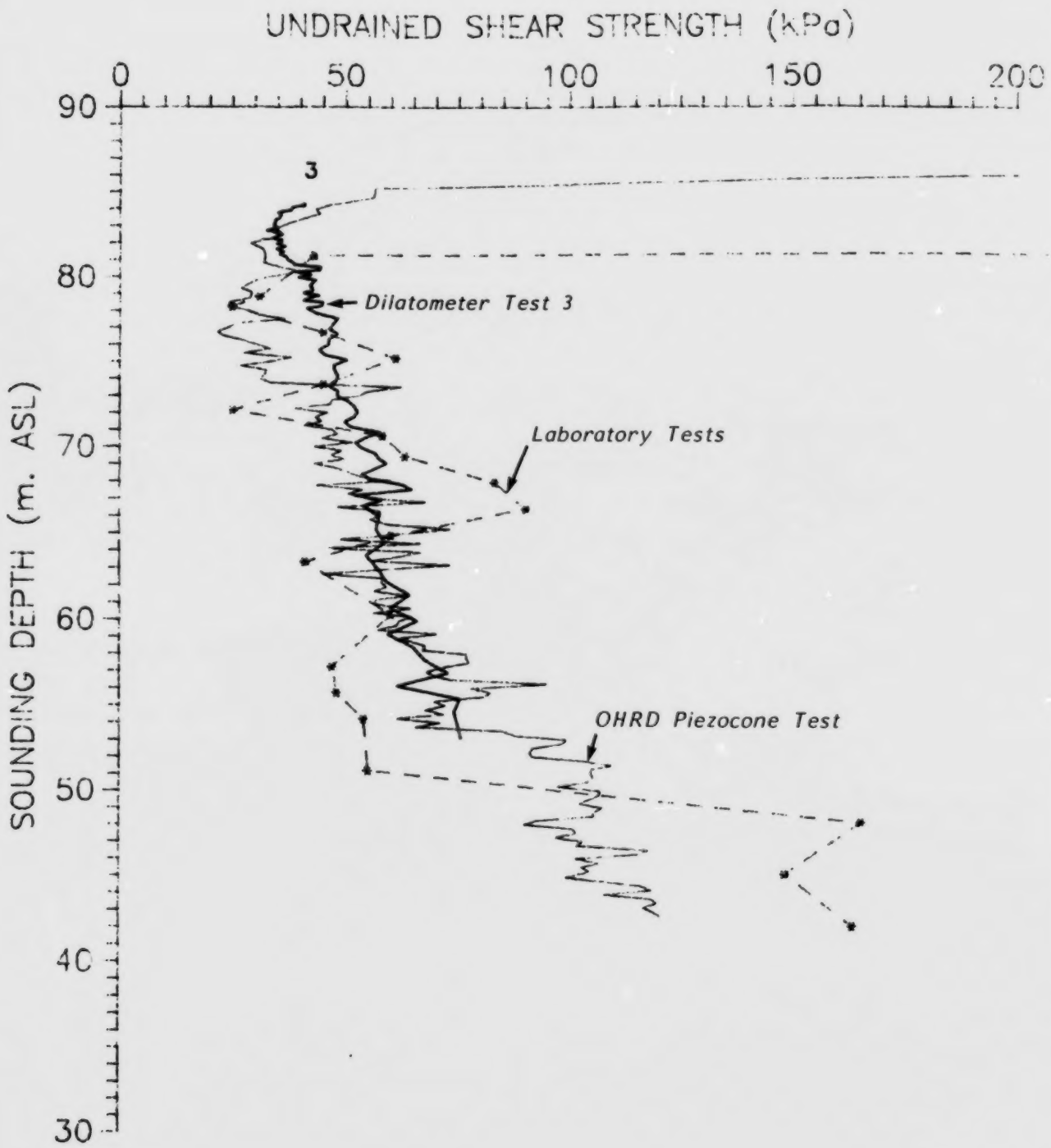


FIGURE 8

COMPARISON OF LABORATORY AND FIELD TESTS AT WABA DAM (1987)

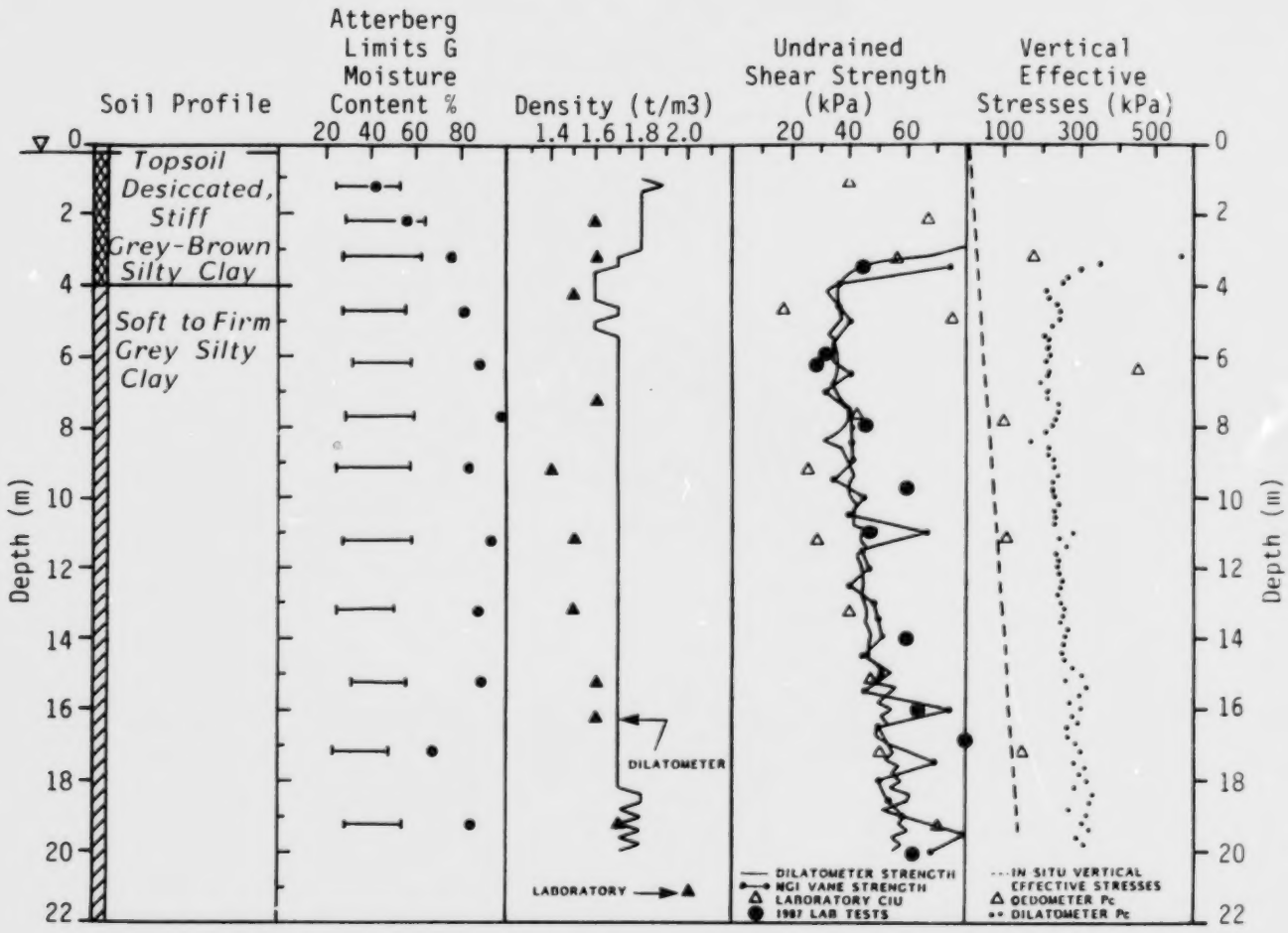


FIGURE 9

COMPARISON OF 1987 LABORATORY AND 1985 TESTS AT WABA DAM

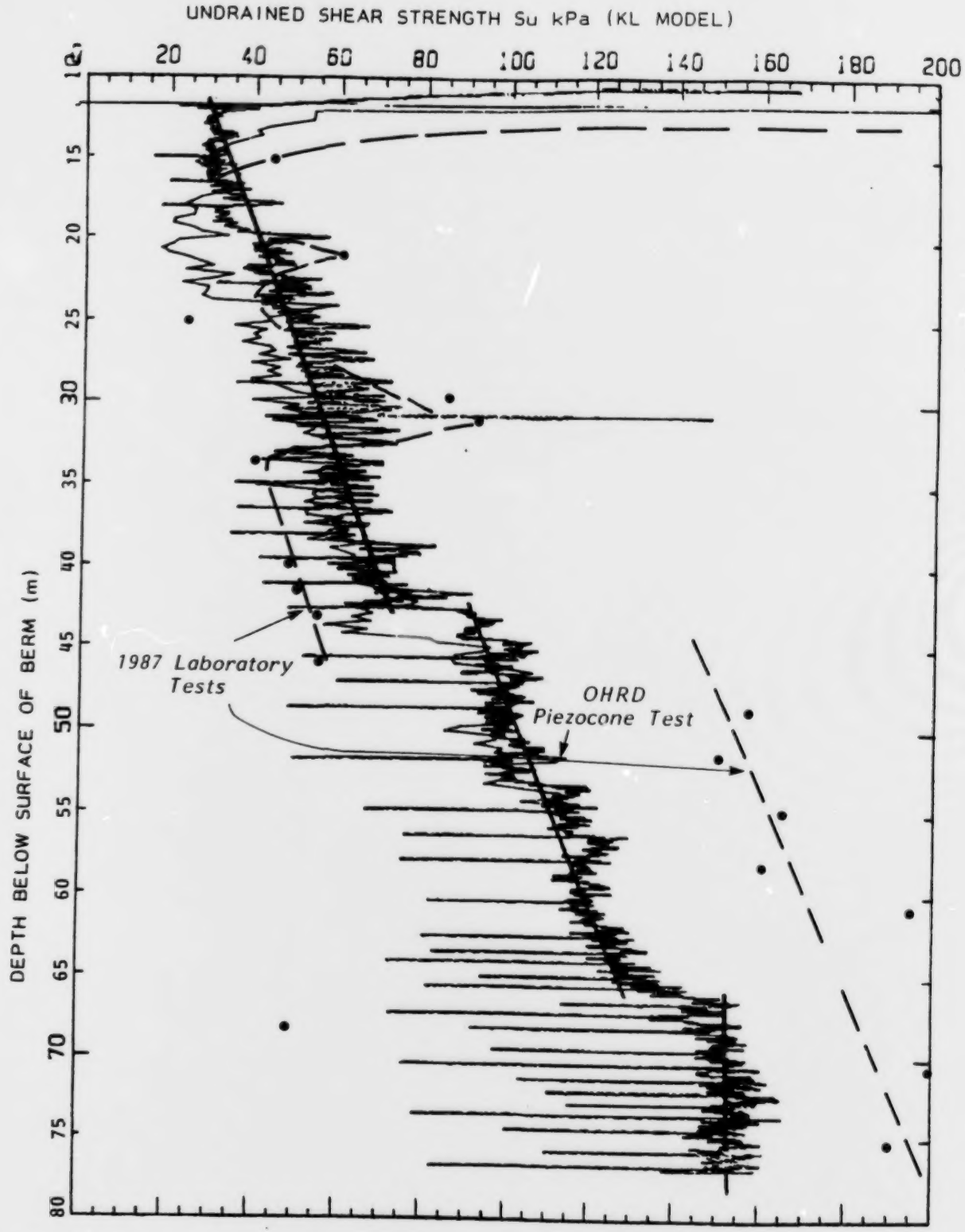


FIGURE 10

NRC PIEZOCONE AND LABORATORY TESTS
COMPARED AT WABA DAM

176182 RD

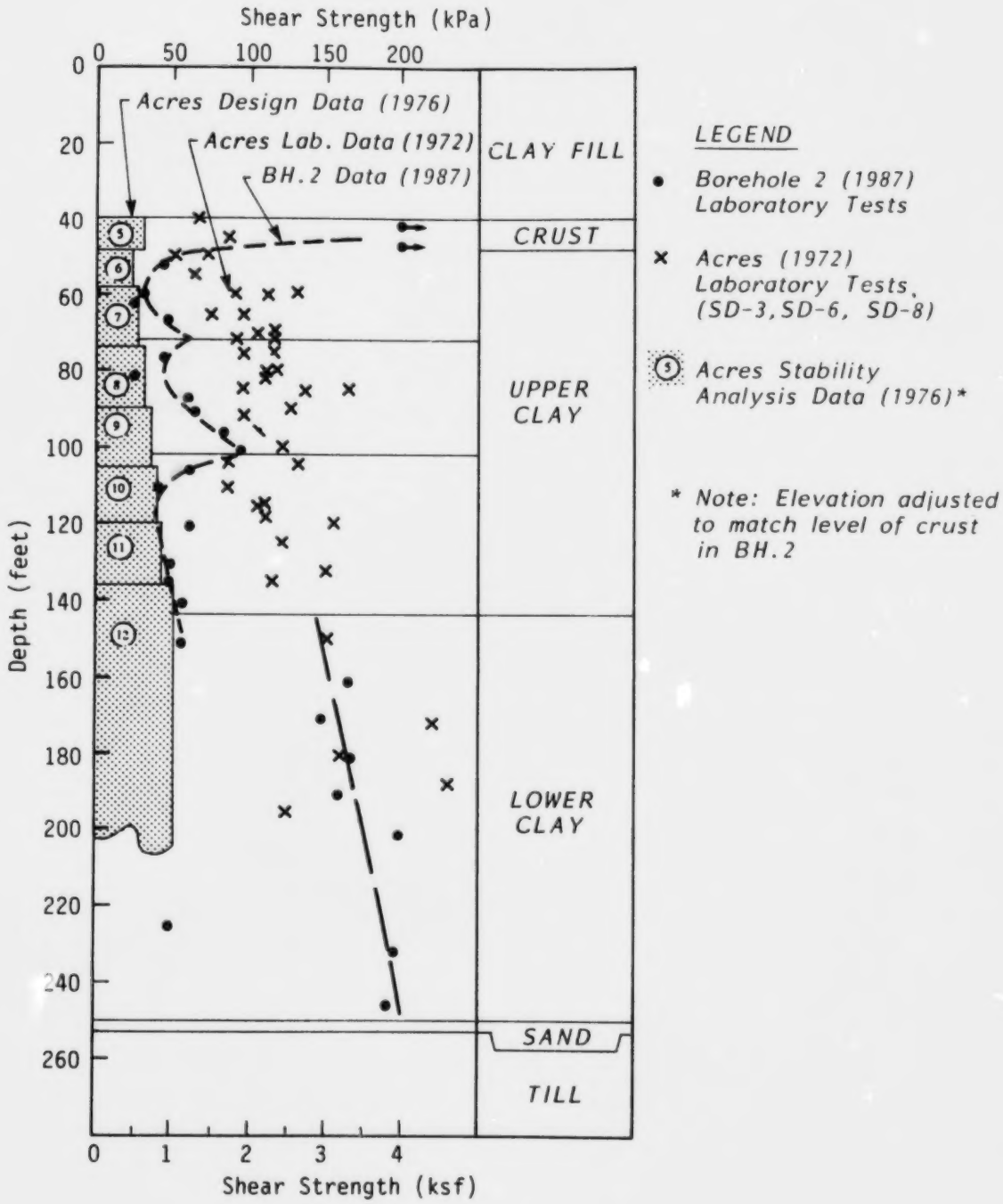


FIGURE 11

1987 MEASUREMENTS VERSUS ORIGINAL DESIGN SHEAR STRENGTHS AT WABA DAM

176176 RD

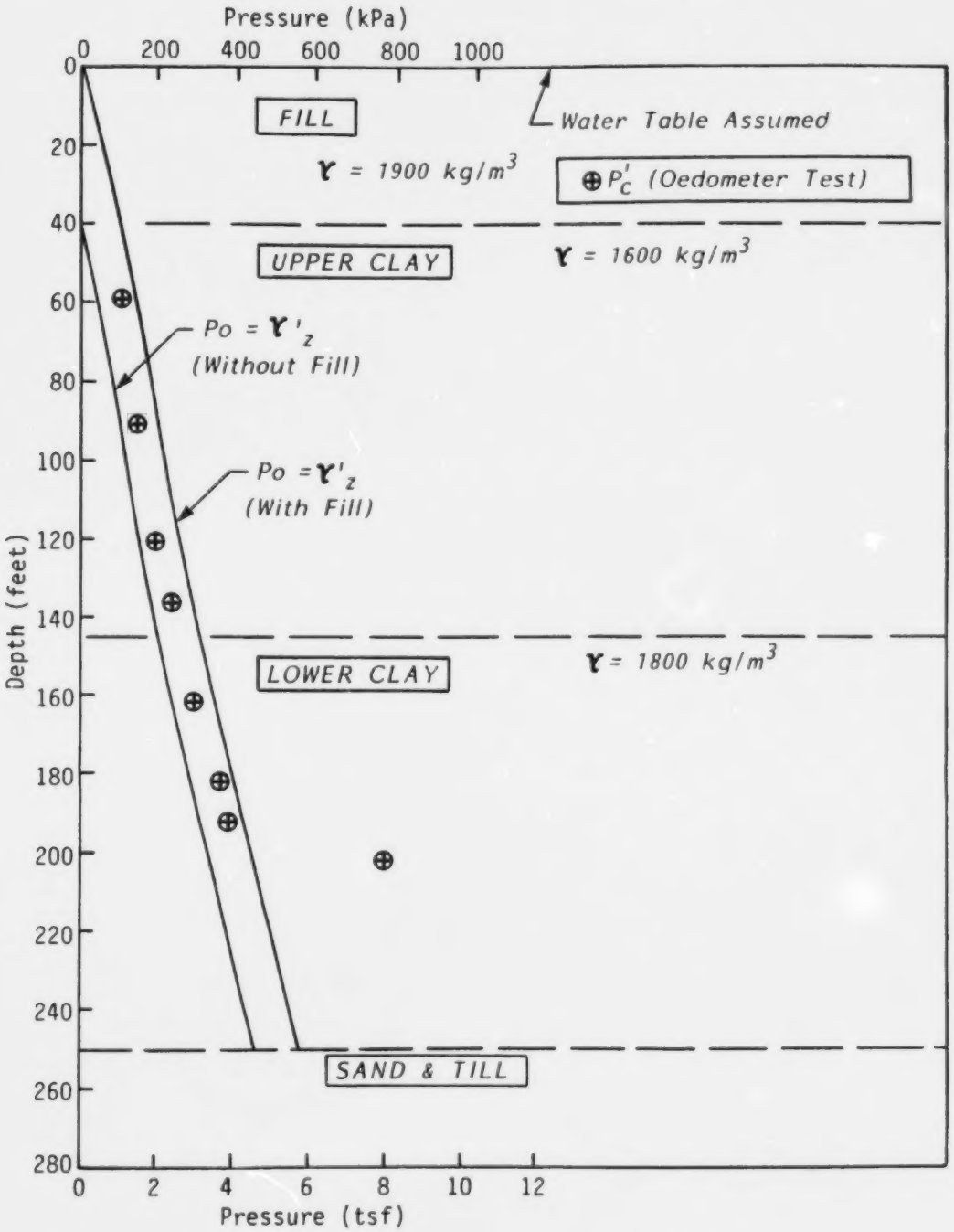


FIGURE 12

WABA DAM - PHASE II
CONSOLIDATION TESTS

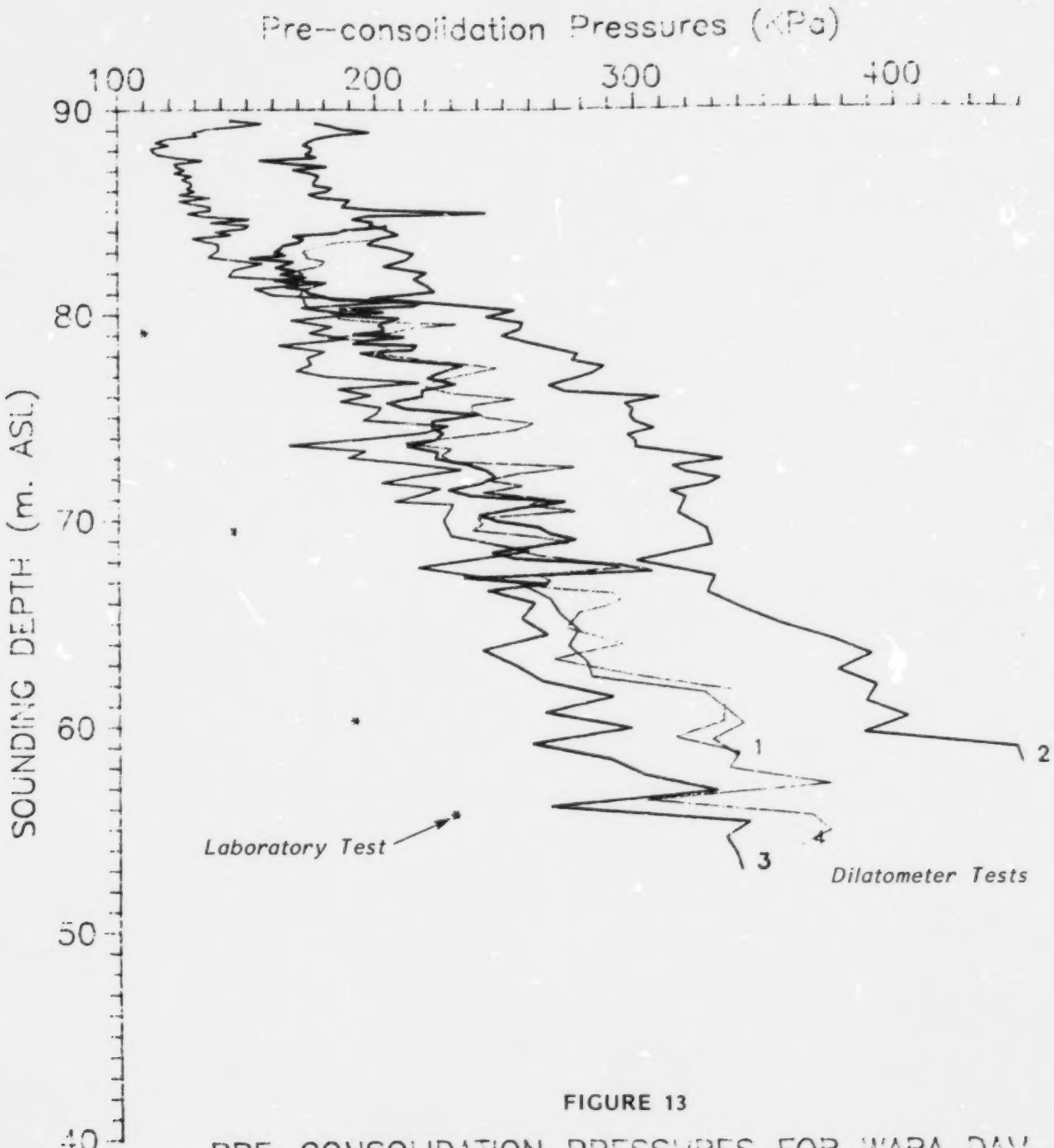


FIGURE 13

PRE-CONSOLIDATION PRESSURES FOR WABA DAM FROM DILATOMETER AND OEDOMETER TESTS

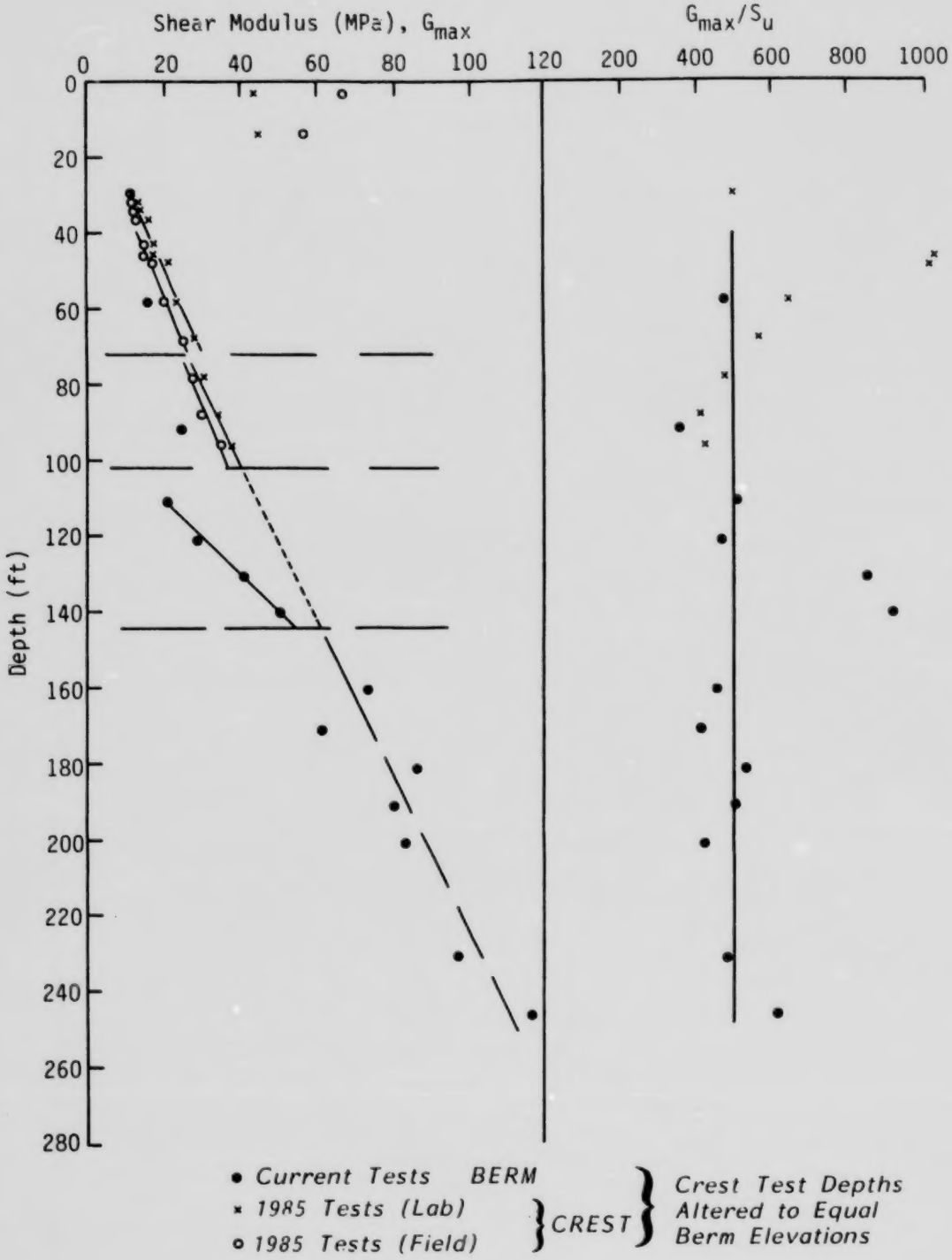


FIGURE 14

WABA DAM: PHASE II
DYNAMIC SHEAR MODULUS TESTS

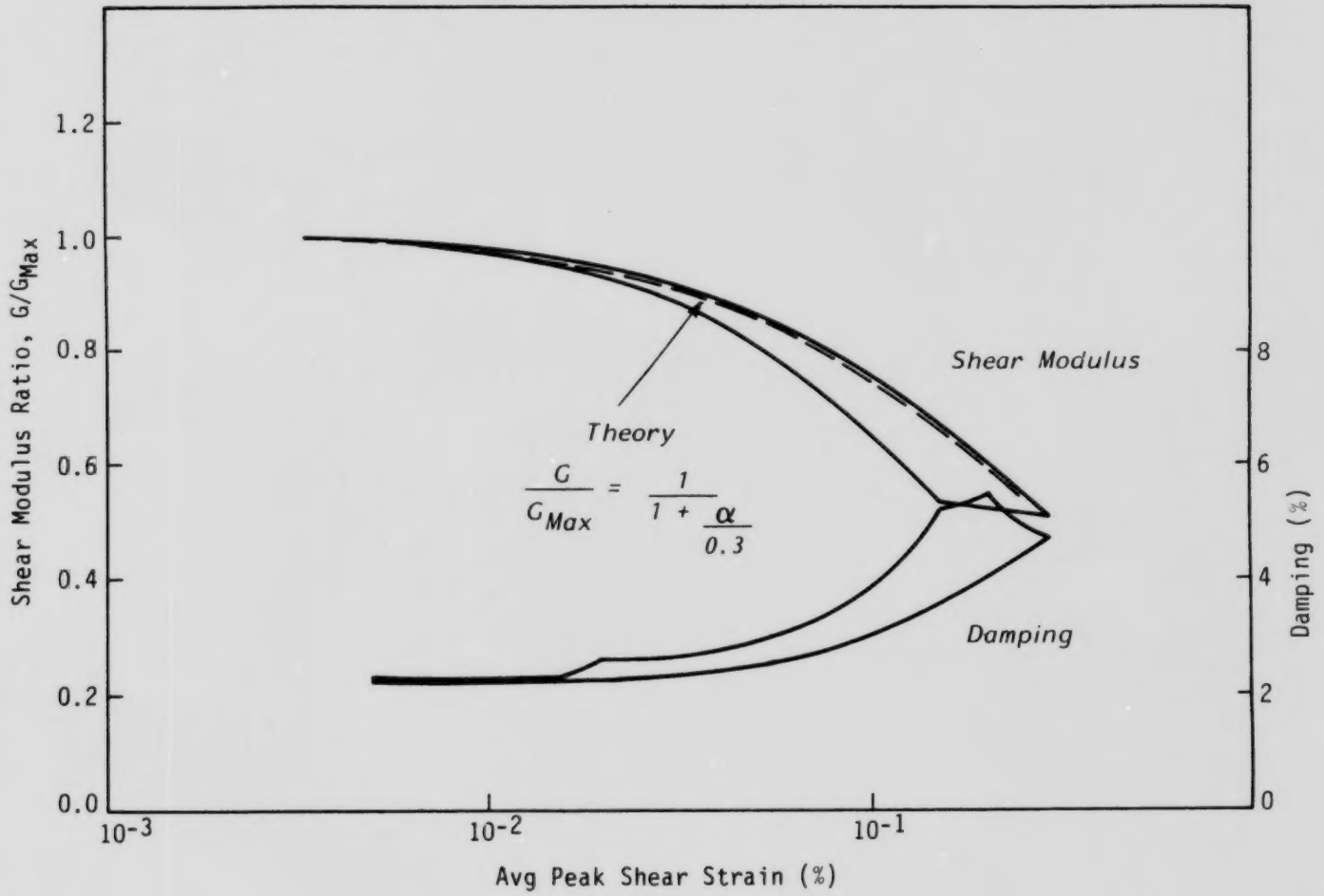


FIGURE 15
WABA DAM (1987)
UPPER CLAY (40'-140')
6 SAMPLES

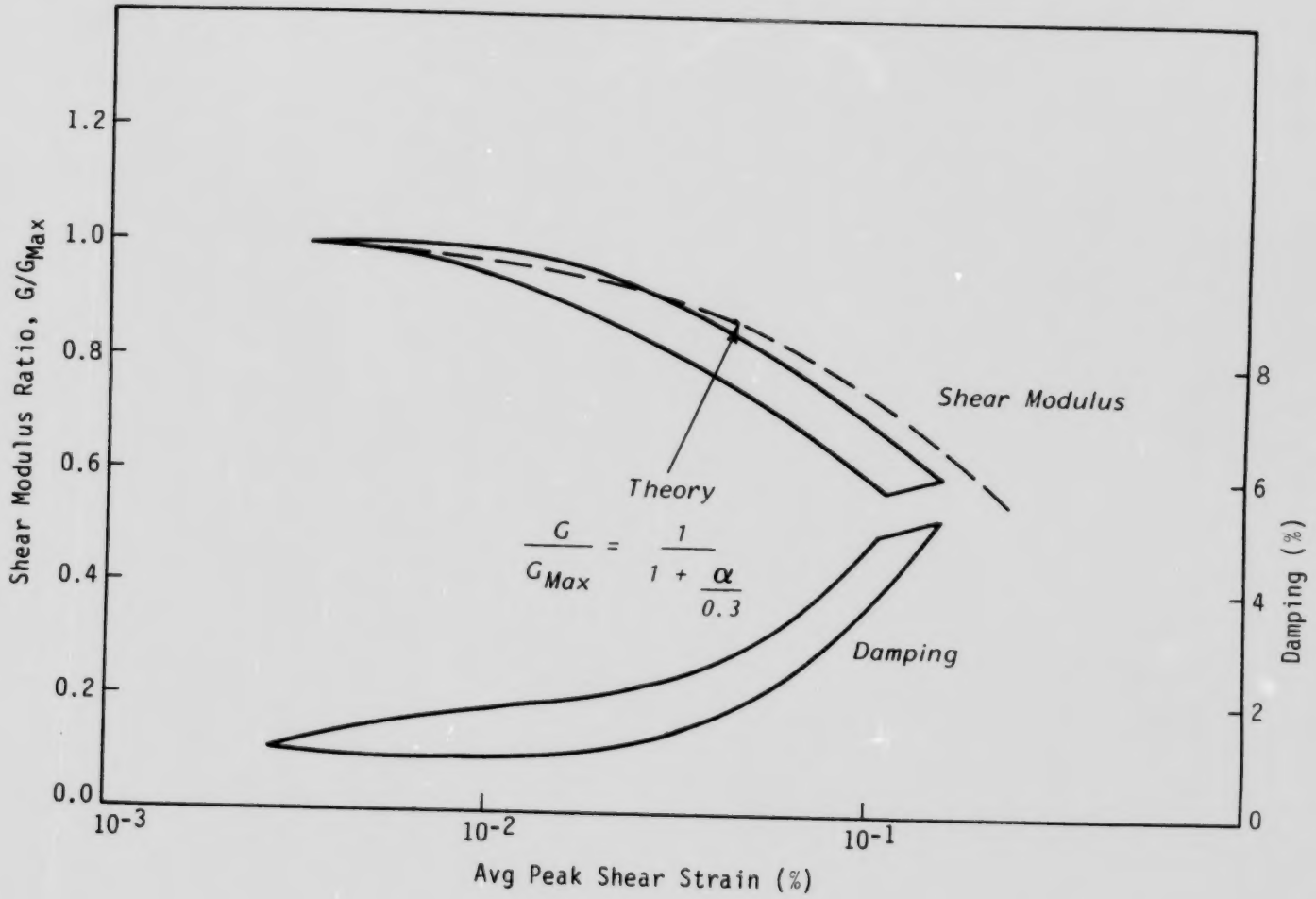


FIGURE 16
WABA DAM (1987)
LOWER CLAY (140'-250')
7 SAMPLES

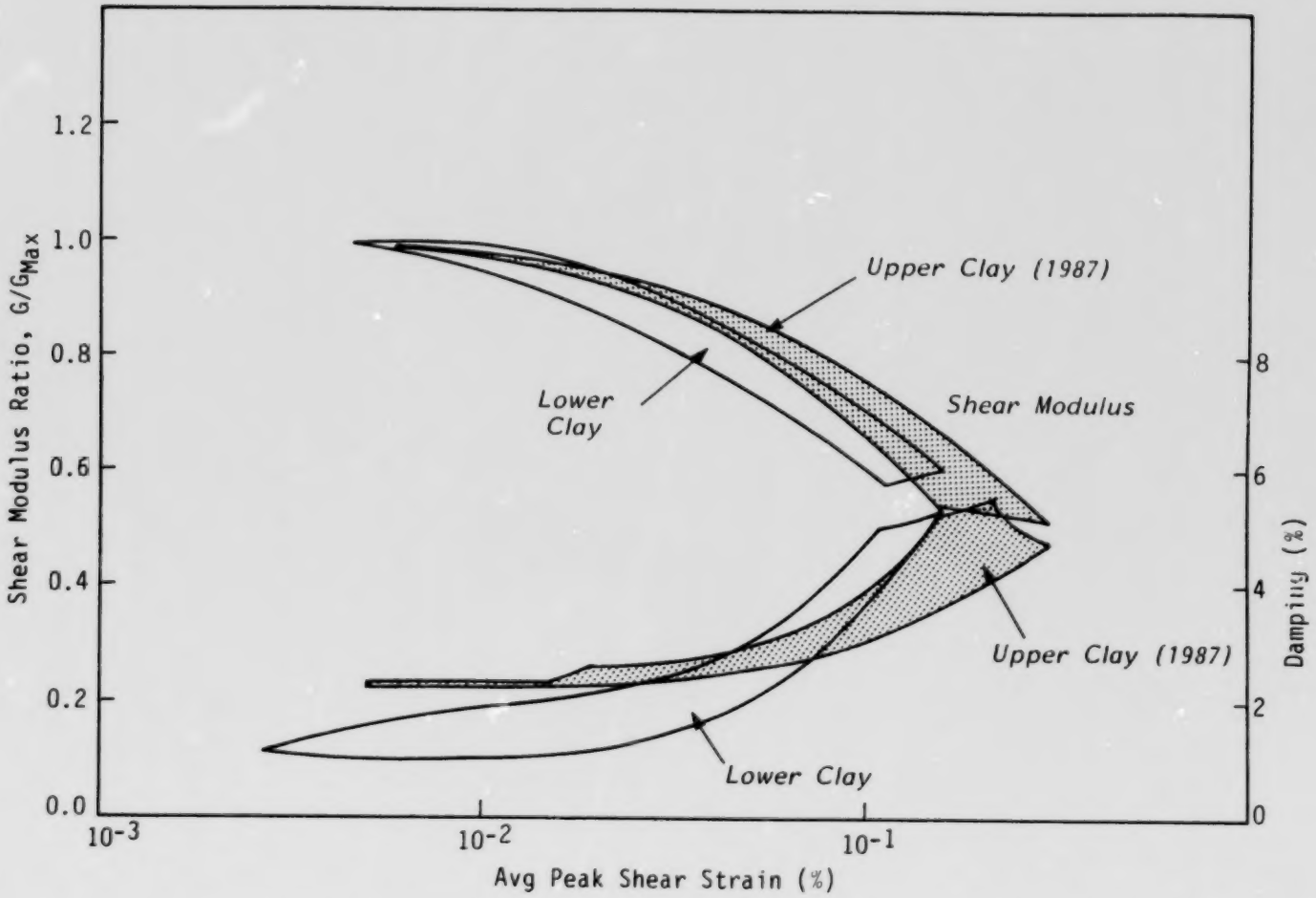


FIGURE 17
COMPARISON OF UPPER AND
LOWER CLAY RESULTS

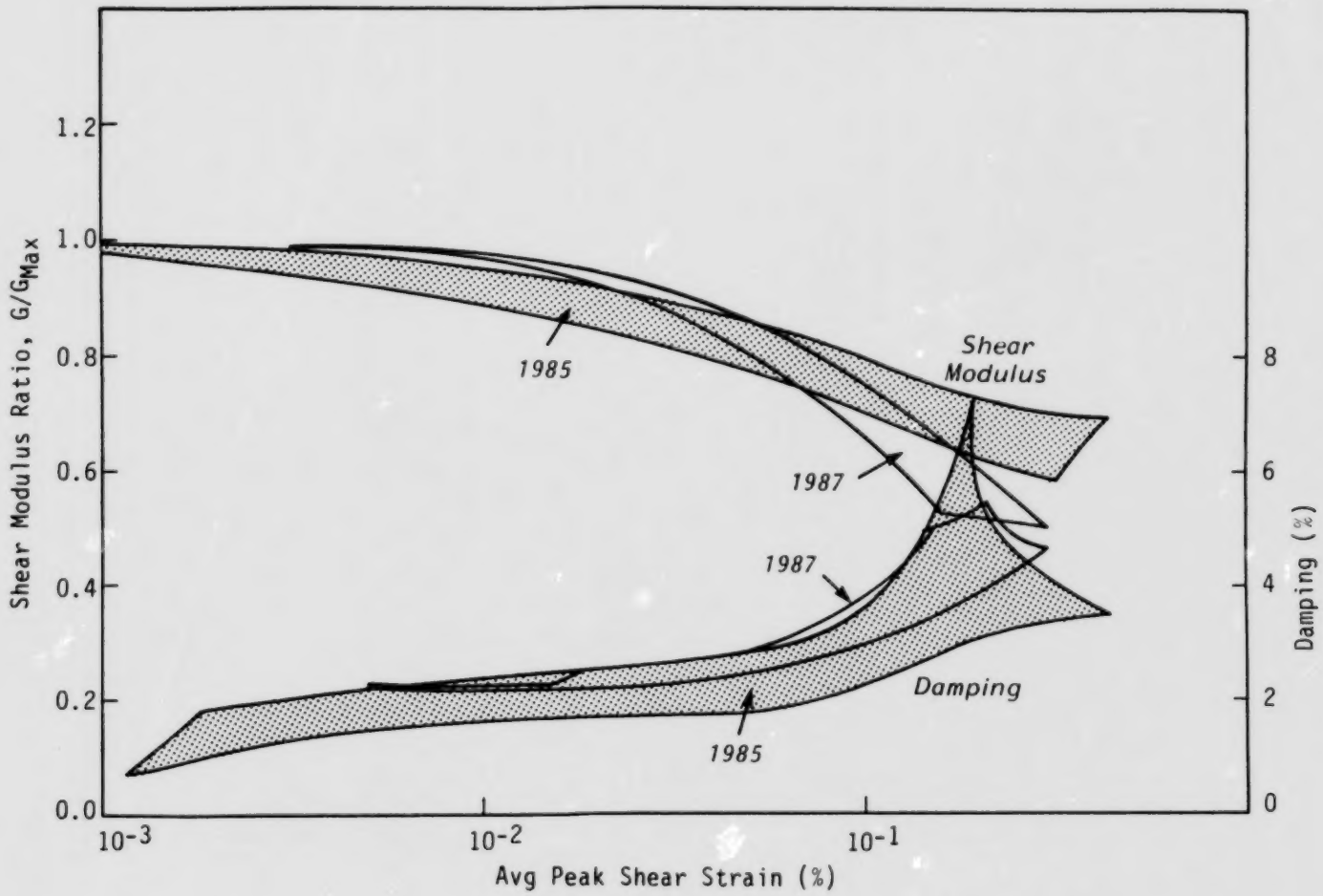


FIGURE 18
COMPARISON OF 1985 AND 1987
TESTS ON UPPER CLAY

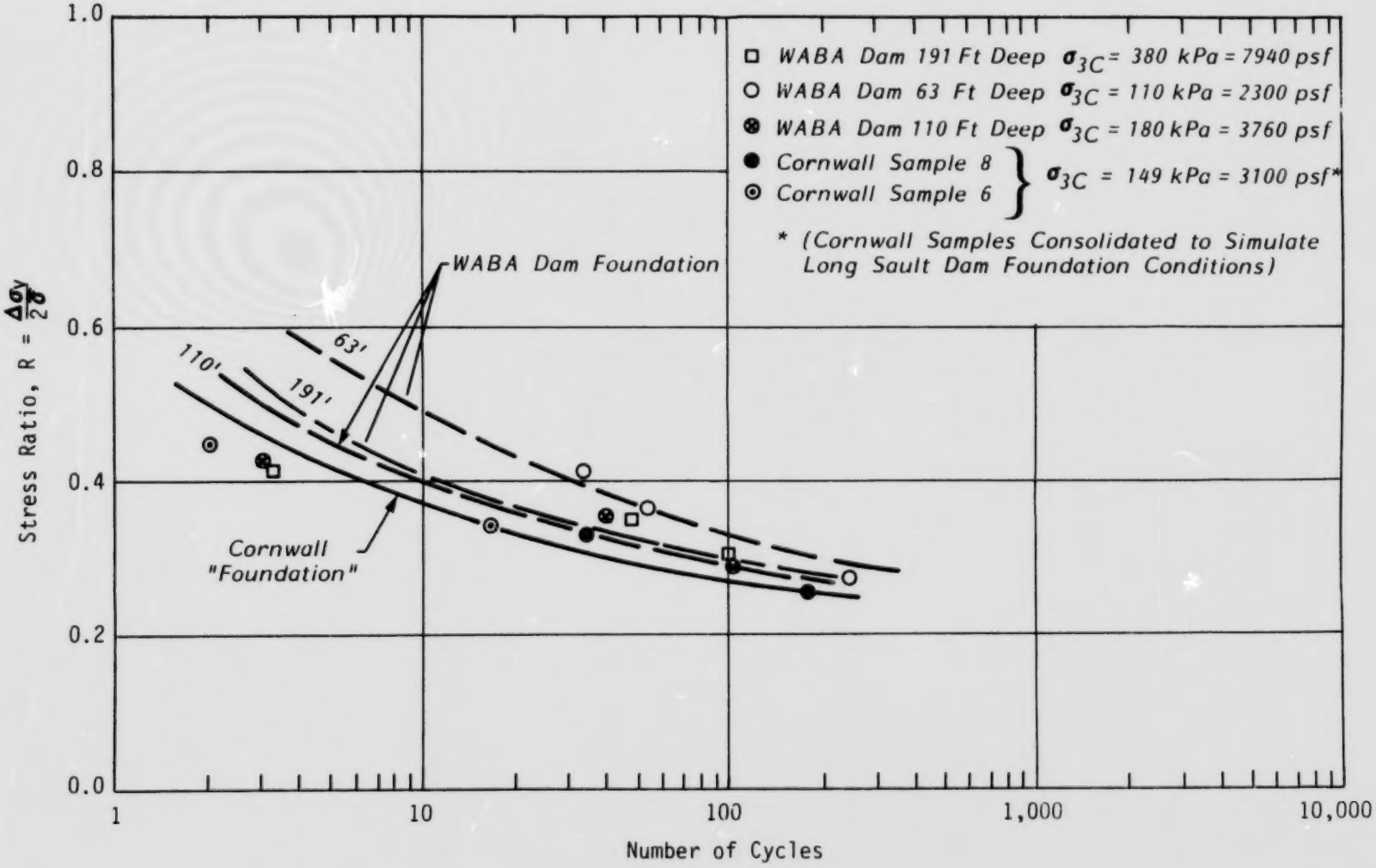
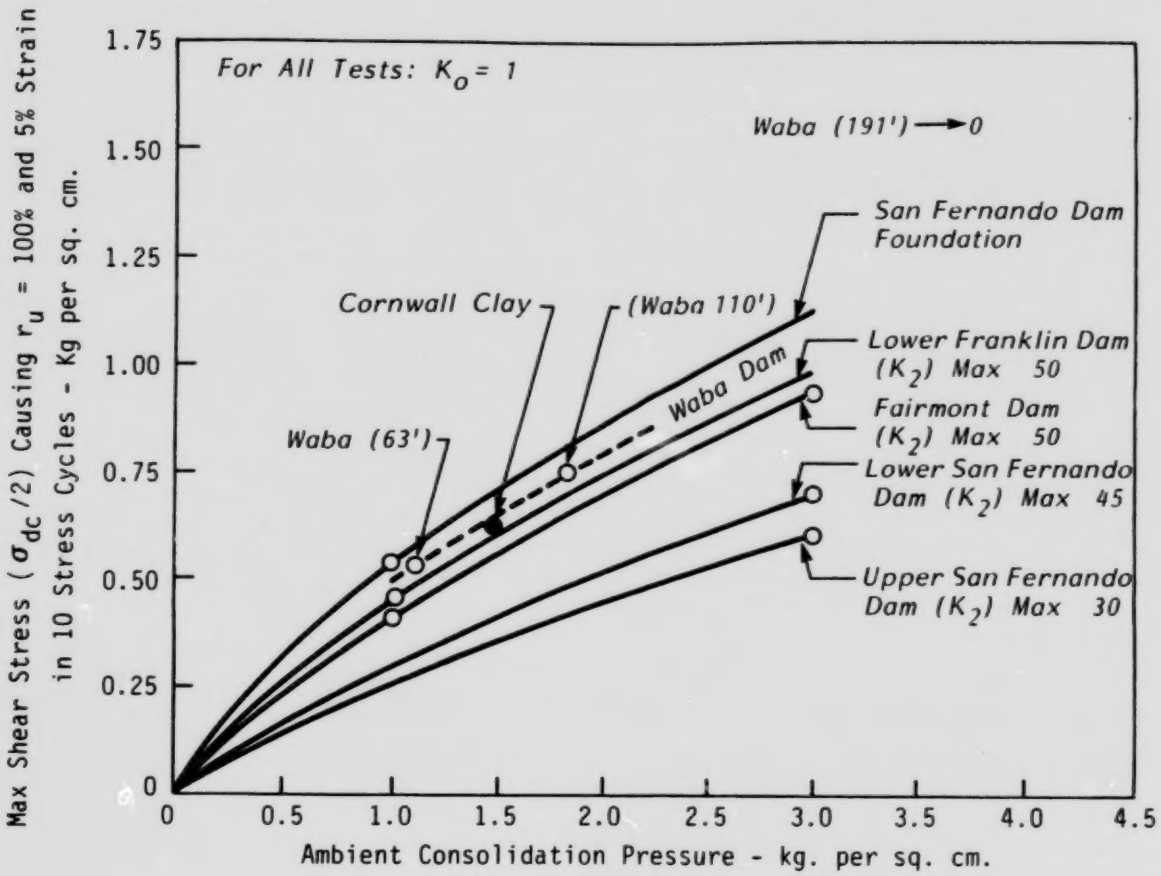


FIGURE 19
CYCLIC STRENGTH OF DYKE FOUNDATION MATERIAL

87-329 176273 RD



Plot from Seed, Makdisi and DeAlba (1978)

FIGURE 20

COMPARISON OF CYCLIC STRENGTH OF SENSITIVE CLAY TO CYCLIC STRENGTH OF WEAK EMBANKMENT DAM MATERIALS

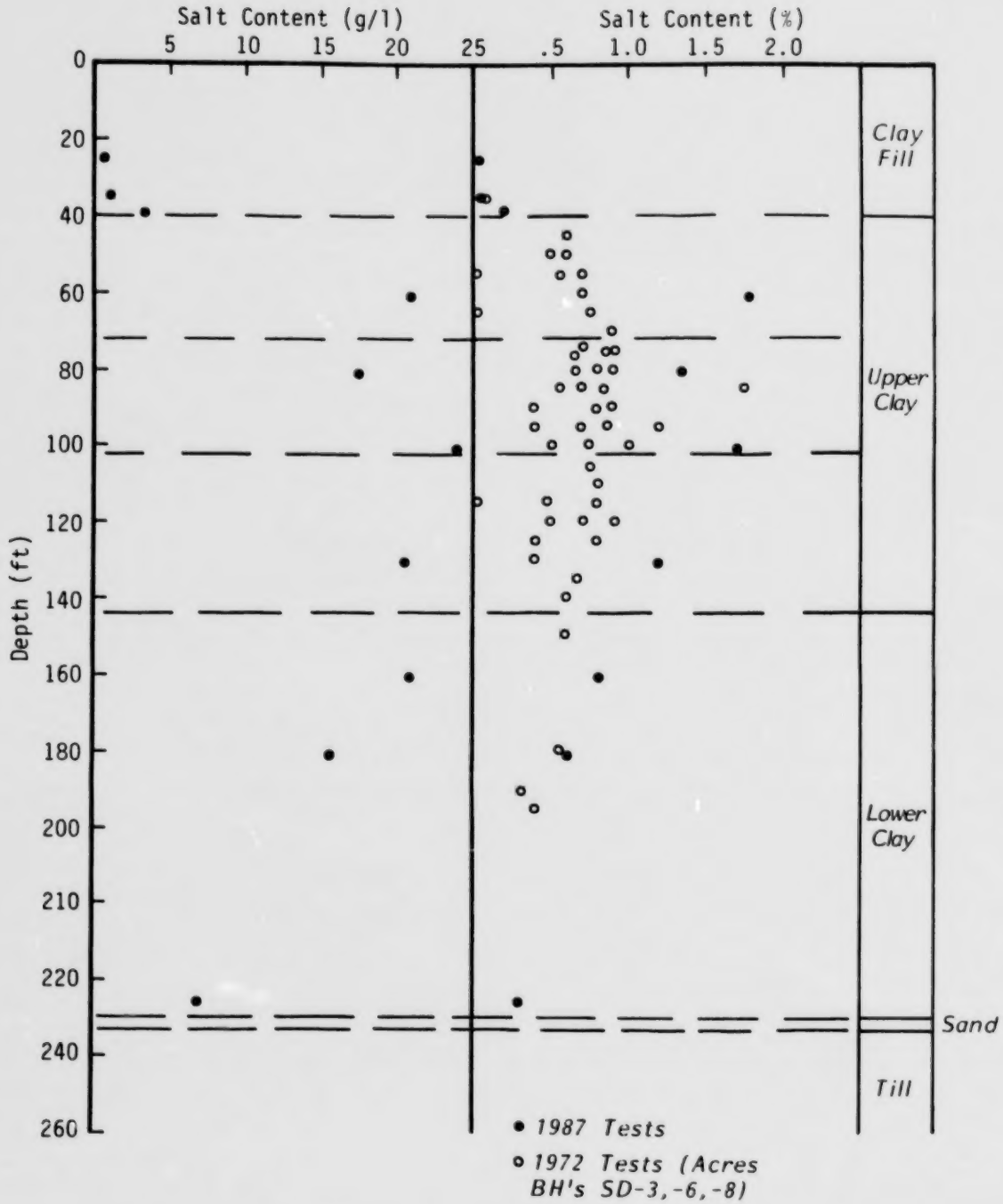


FIGURE 21

SALT CONTENT OF WABA DAM CLAY SAMPLES

APPENDIX A

WABA DAM - PHASE II

CONSOLIDATED UNDRAINED TRIAXIAL (CIU) TESTS

WABA DAM - UNDRAINED TRIAXIAL TESTS (CIU)

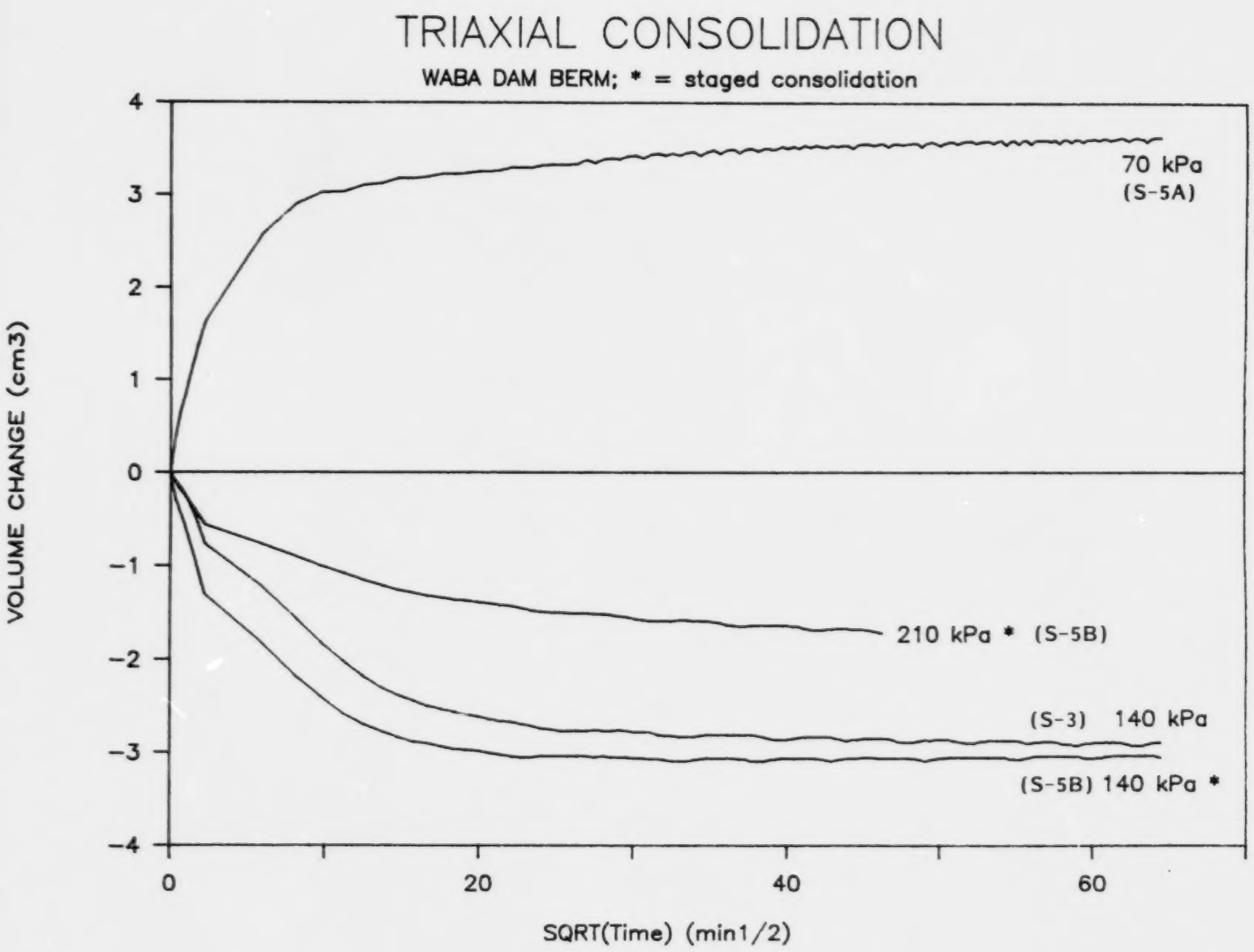
SOIL	c'	ϕ'	DETAILS
BERM	21.5 kPa	30.5°	FIGURES A1 - A5
UPPER CLAY	30.0 kPa 8.0 kPa	27.5° 22.5° "overstressed"	FIGURES A6 - A14
LOWER CLAY	22.0 kPa	30.0°	FIGURES A15 - A19

CIU TESTS - CREST FILL

SAMPLE NO.	SAMPLE DEPTH (ft)	CONSOLIDATION PRESSURE (kPa)	BULK DENSITY (kg/m^3)	WATER CONTENT (%)
5A	25	70	1900	33.0
3	15	140	2002	30.5
5B	25	210*	1943	30.6

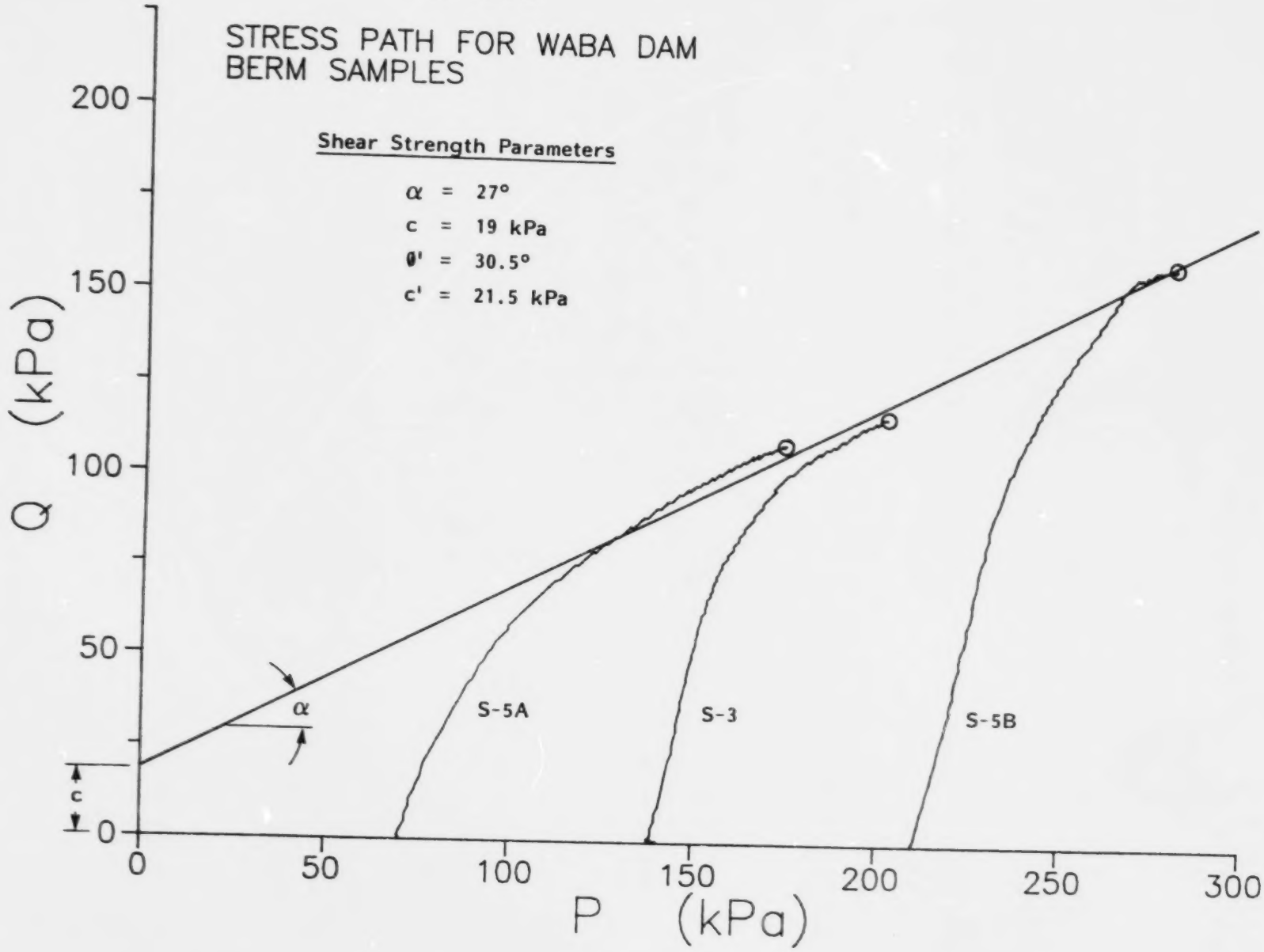
*STAGED CONSOLIDATION

FIGURE A1



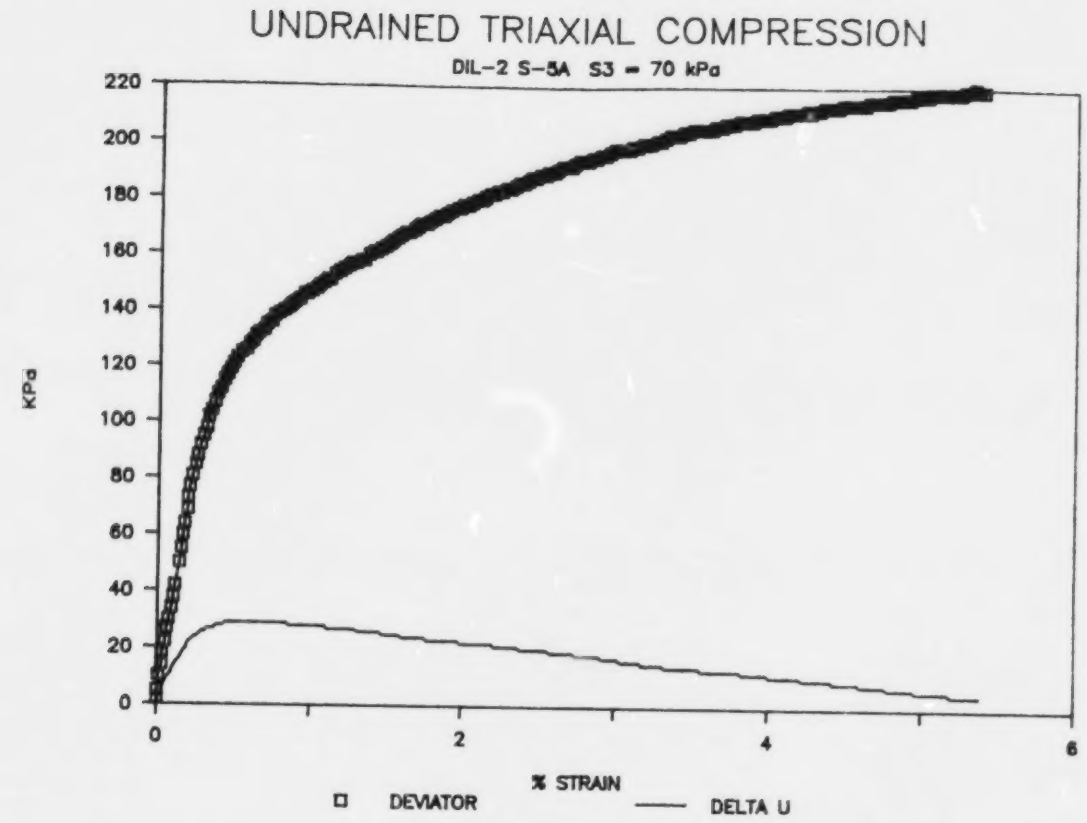
176207 RD
87-329

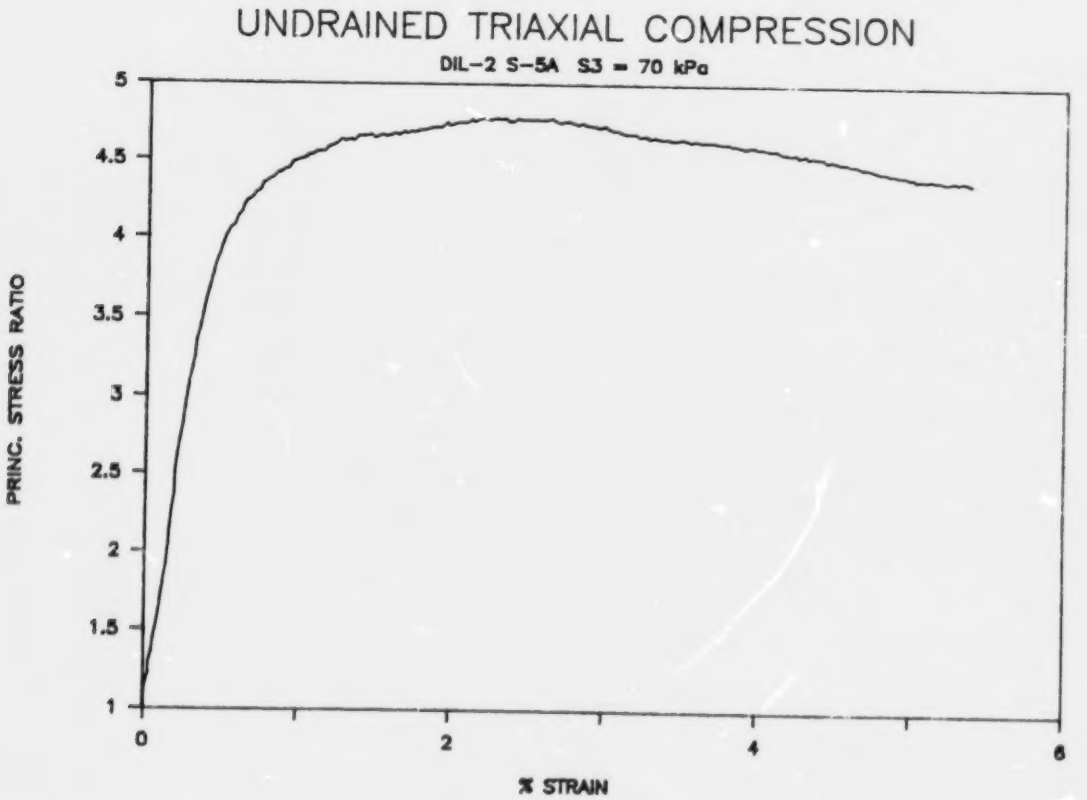
FIGURE A2



87-329
176239 RD

FIGURE A3

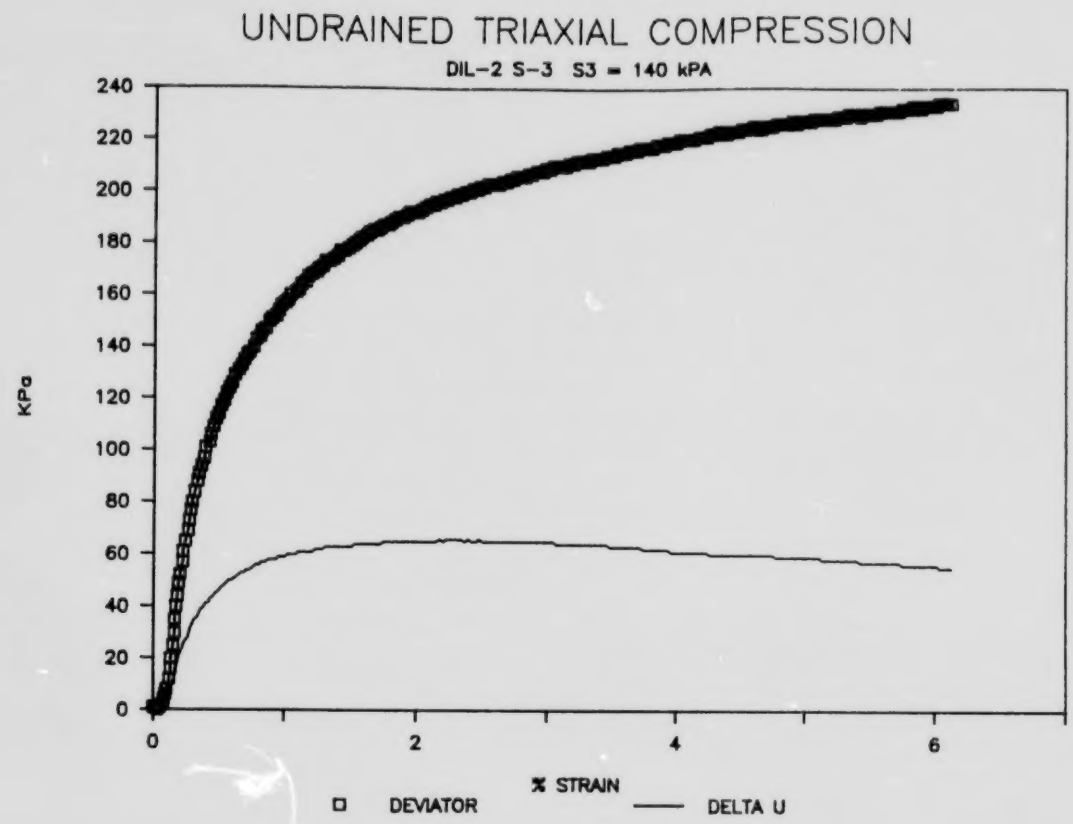


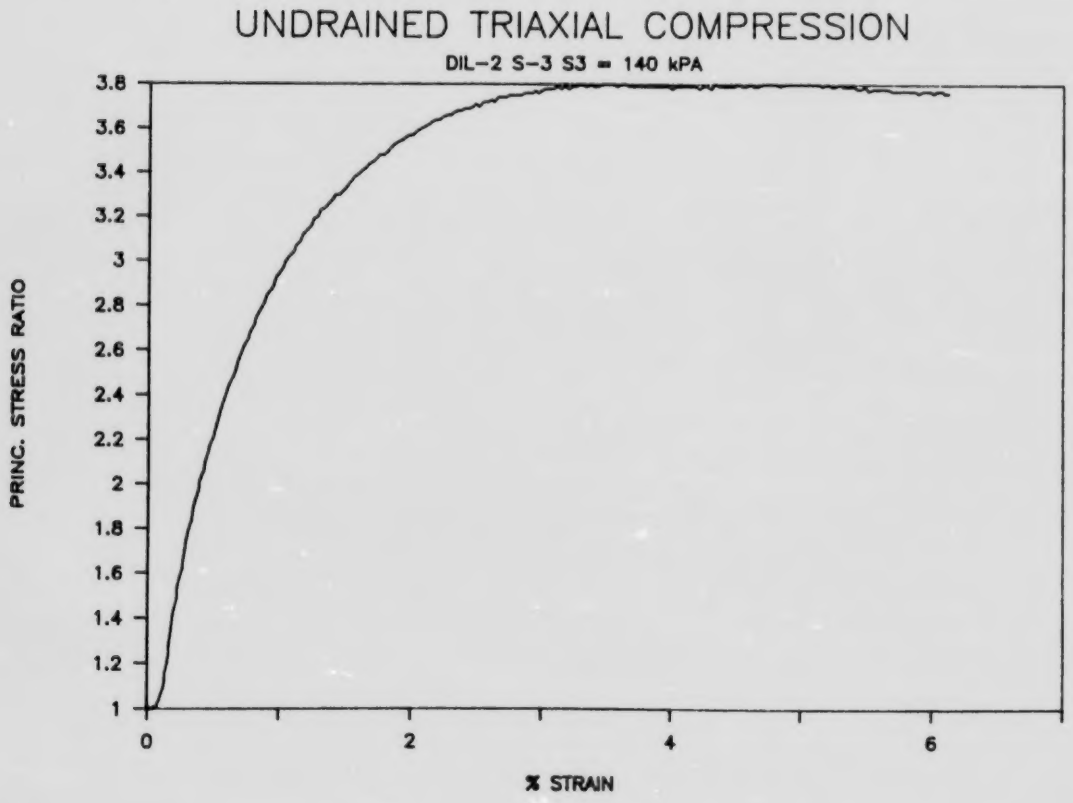


176208 RD

87-329

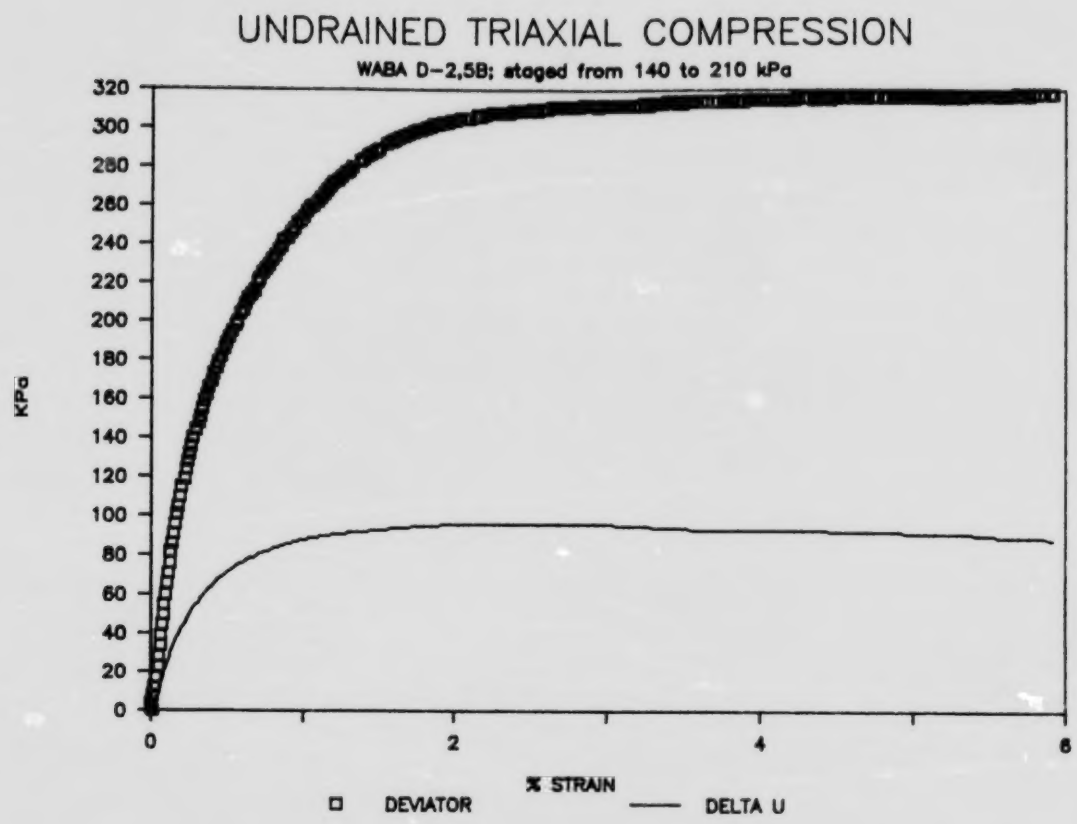
FIGURE A4

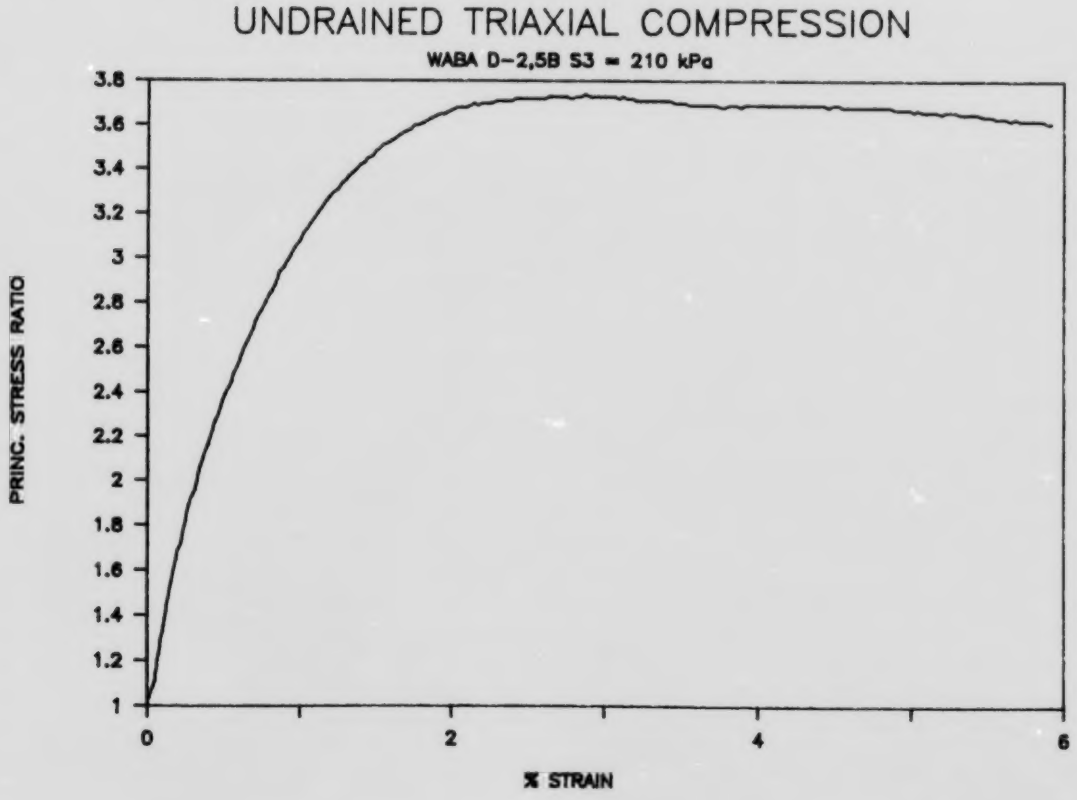




176209 RD

FIGURE A5





176210 RD

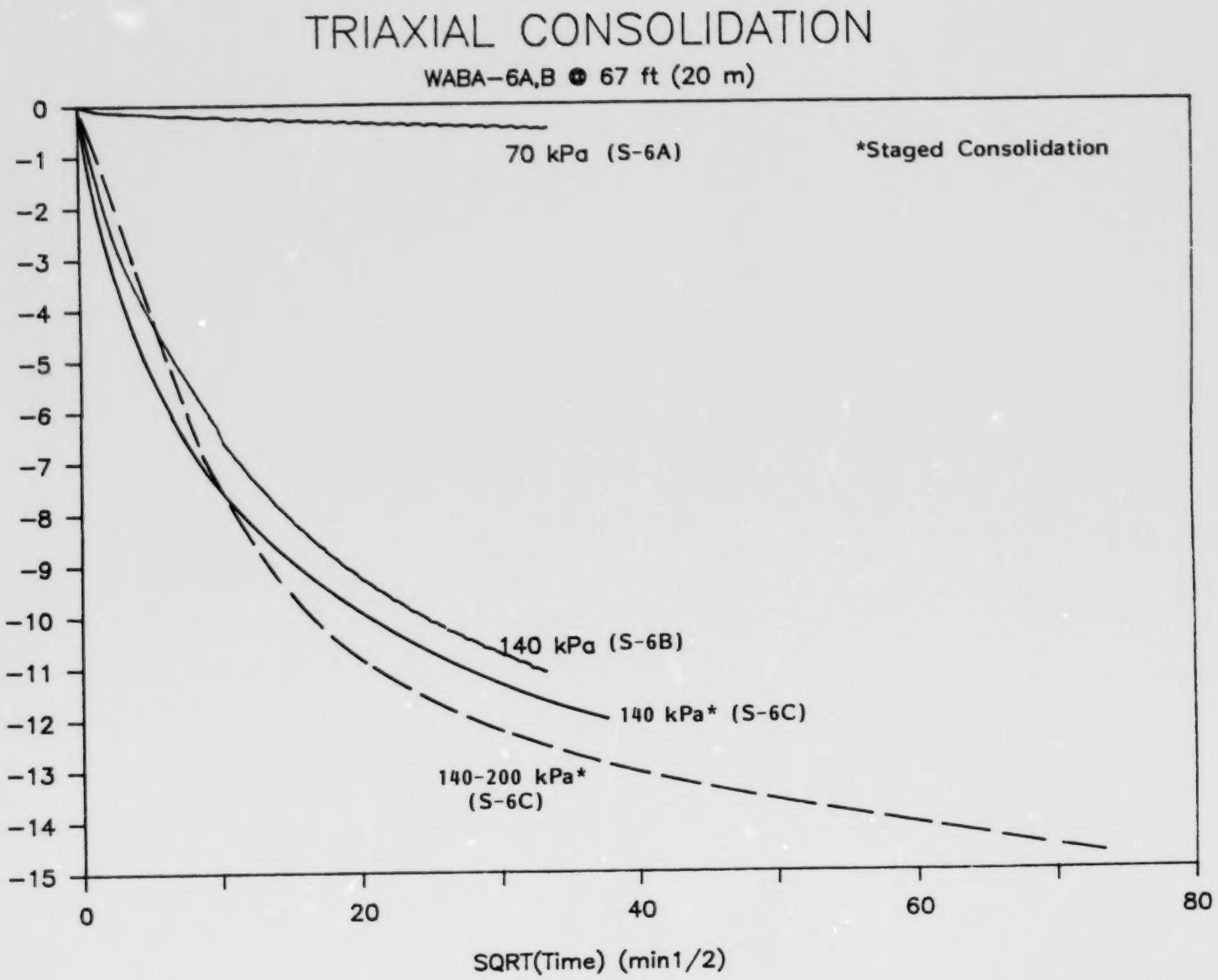
CIU TESTS - UPPER CLAY

SAMPLE NO.	SAMPLE DEPTH (ft)	CONSOLIDATION PRESSURE (kPa)	BULK DENSITY (kg/m^3)	WATER CONTENT (%)
6A	67.0	70	1582	72.5
6B	67.0	140	1582	72.5
6C	67.0	200*	1576	72.5
6D	67.4	40	1531	78.7
6E	67.4	70	1531	78.7
6F	67.4	100	1529	78.7

*STAGED CONSOLIDATION

CRAGG 5
GEM 1 DISK

FIGURE A6



176240 RD
87-329

FIGURE A7

TRIAXIAL CONSOLIDATION

WABA–6D,E,F @ 67ft (20 m)

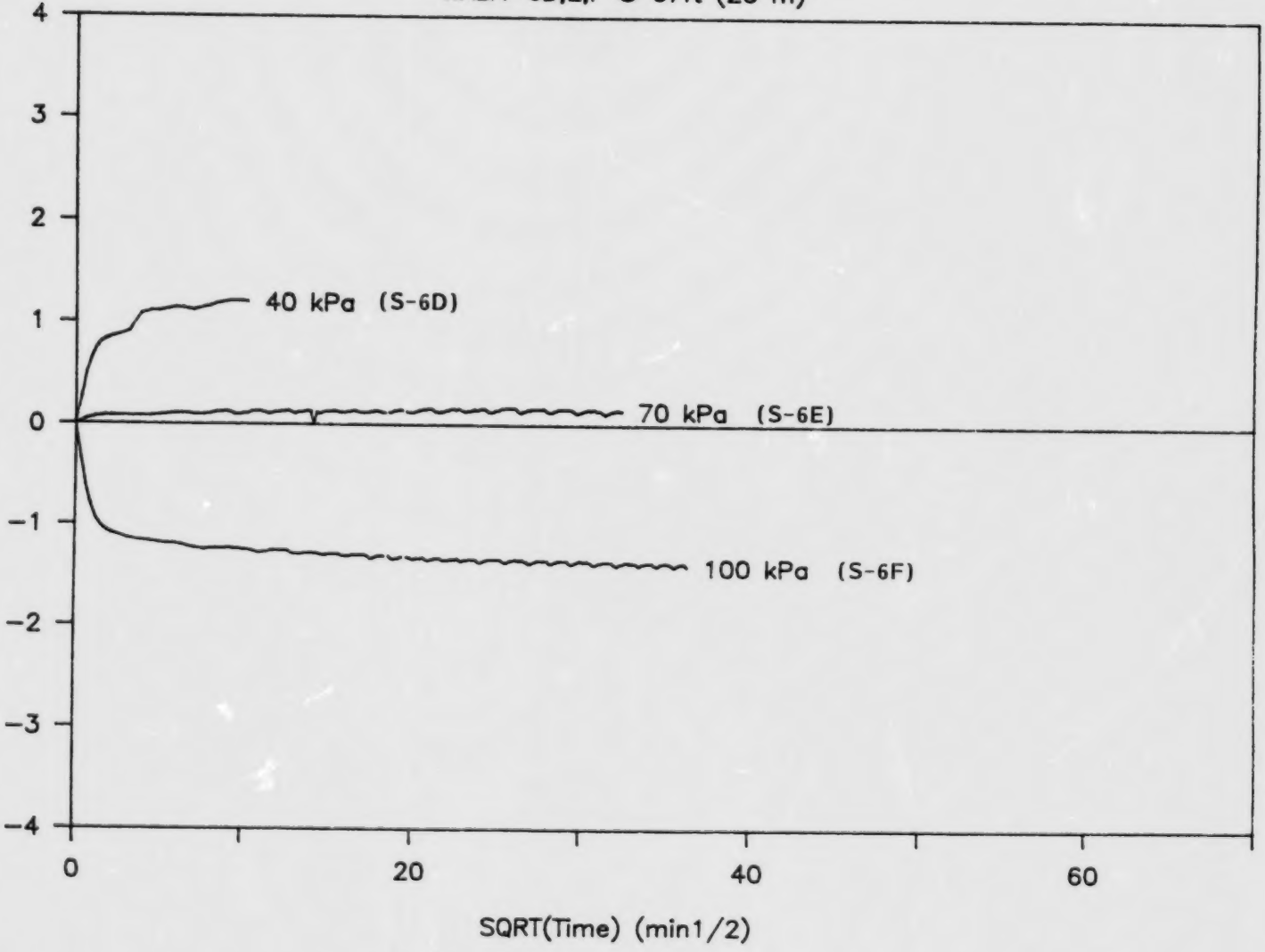
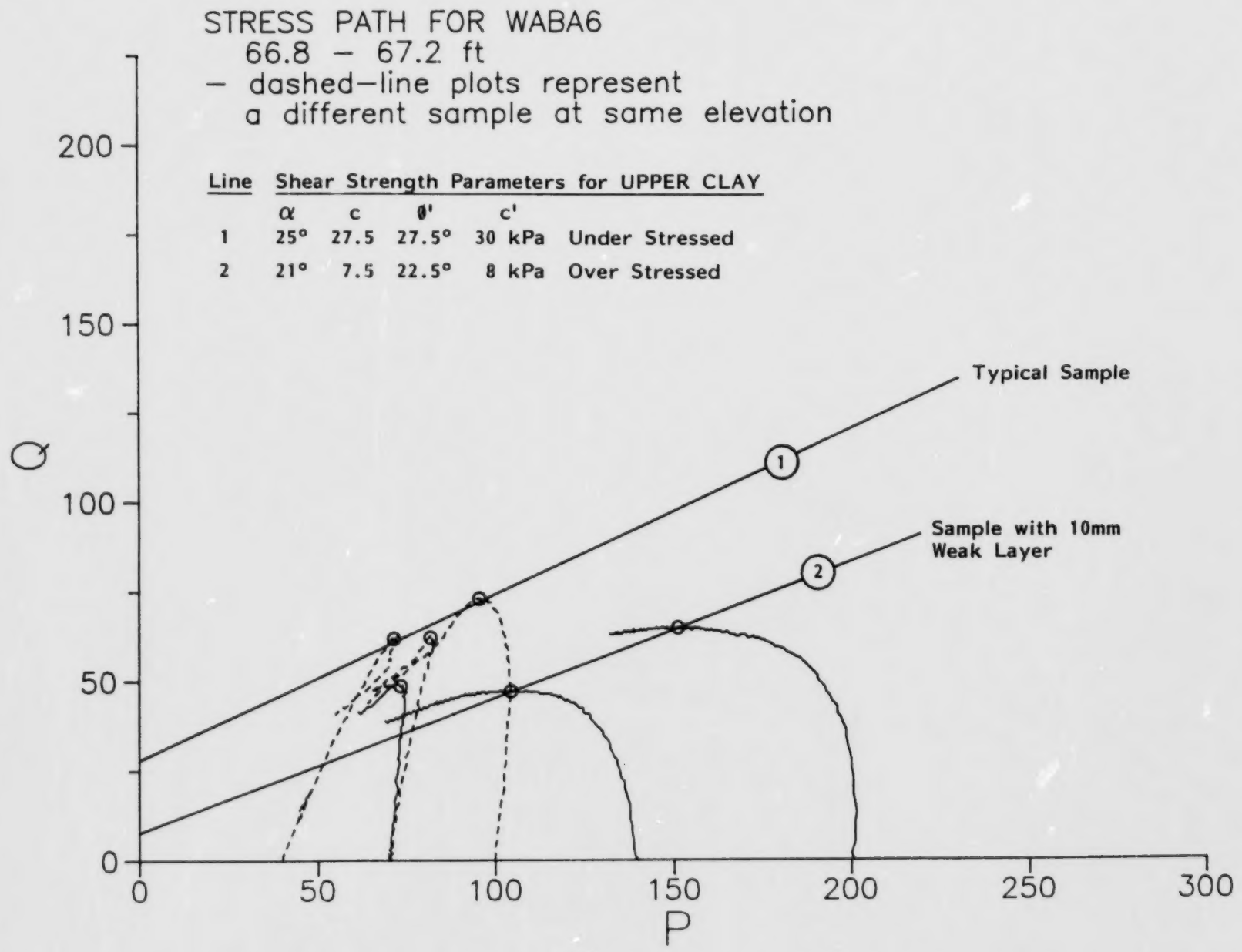
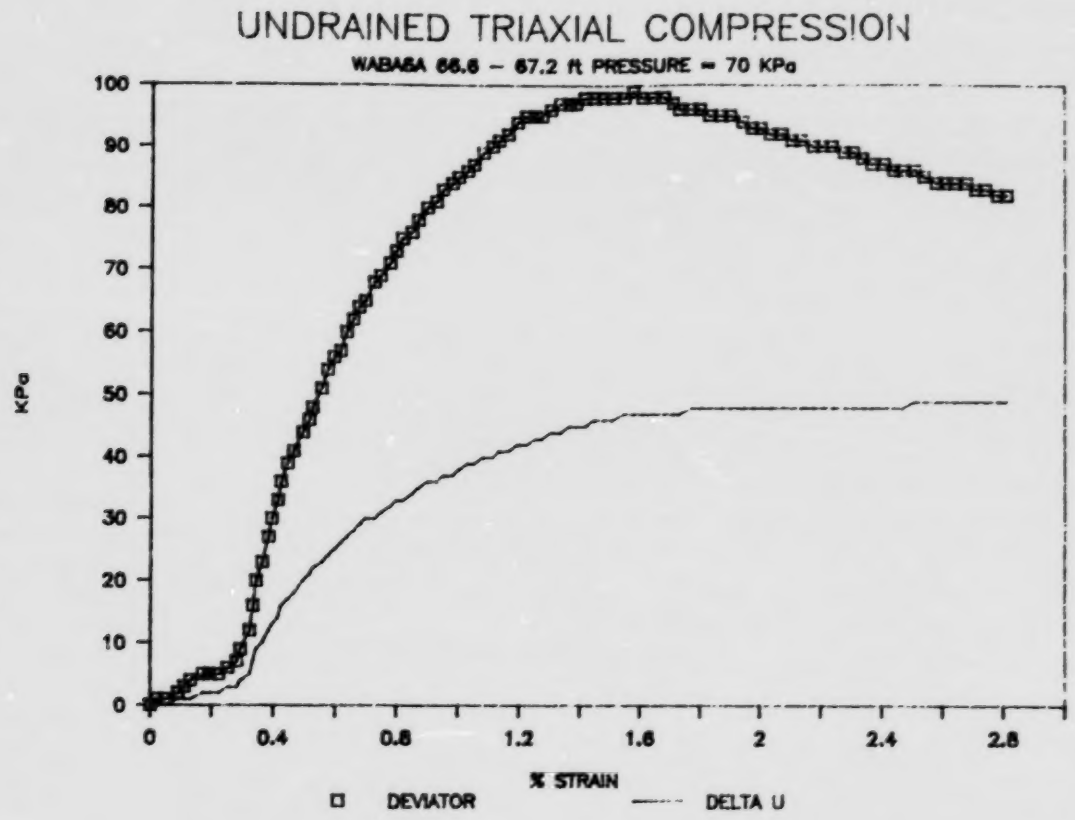


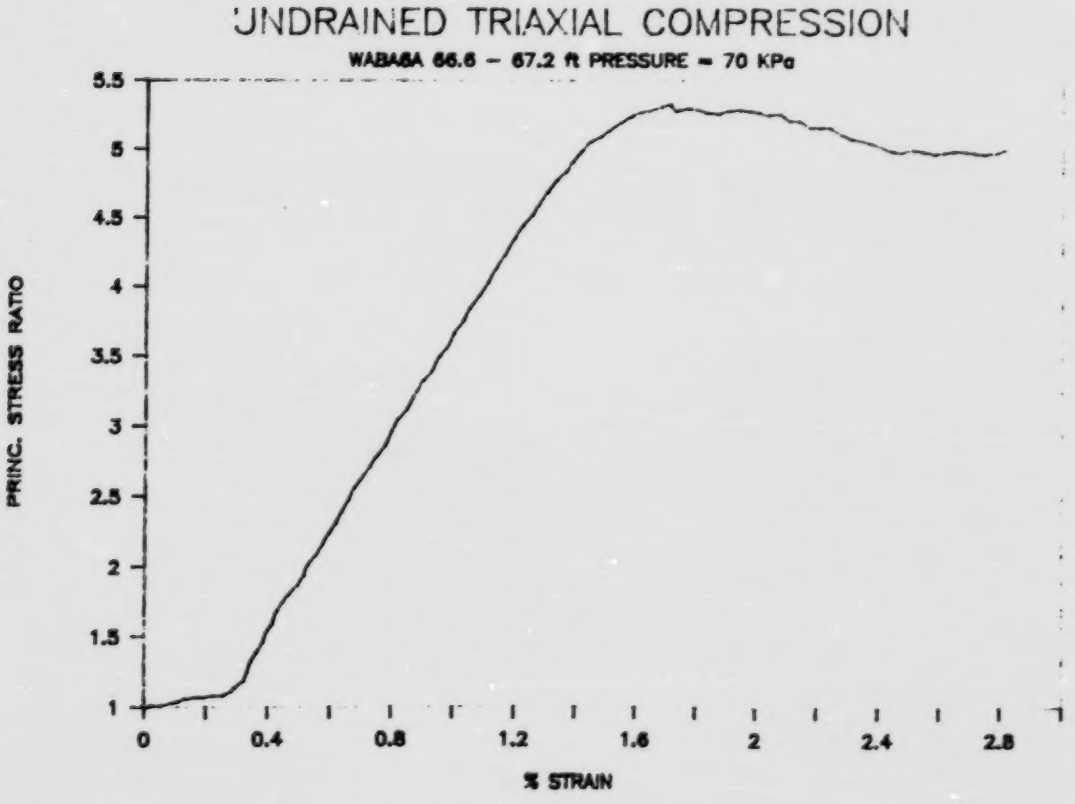
FIGURE A8



87-329
176242 RD

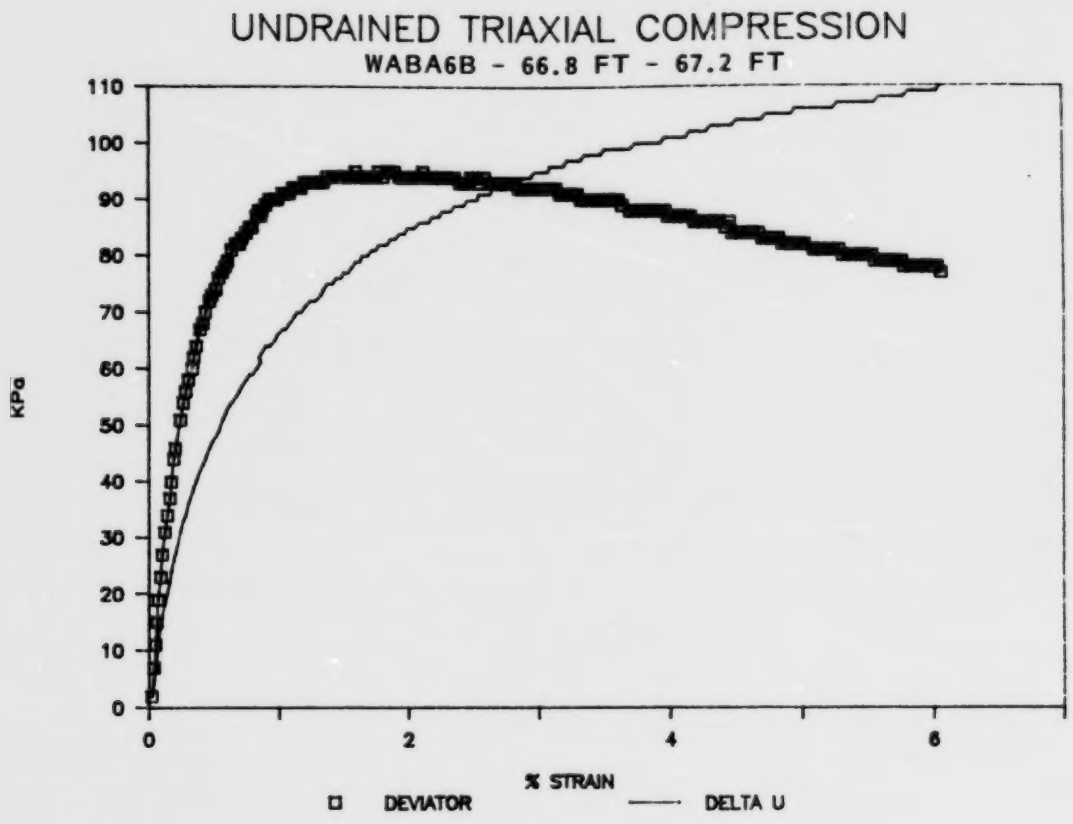
FIGURE A9





176212 RD

FIGURE A10


UNDRAINED TRIAXIAL COMPRESSION
WABA6B - 66.8 FT - 67.2 FT
KPa
110
100
90
80
70
60
50
40
30
20
10
0
0
2
4
6
% STRAIN
DEVIATOR
DELTA U

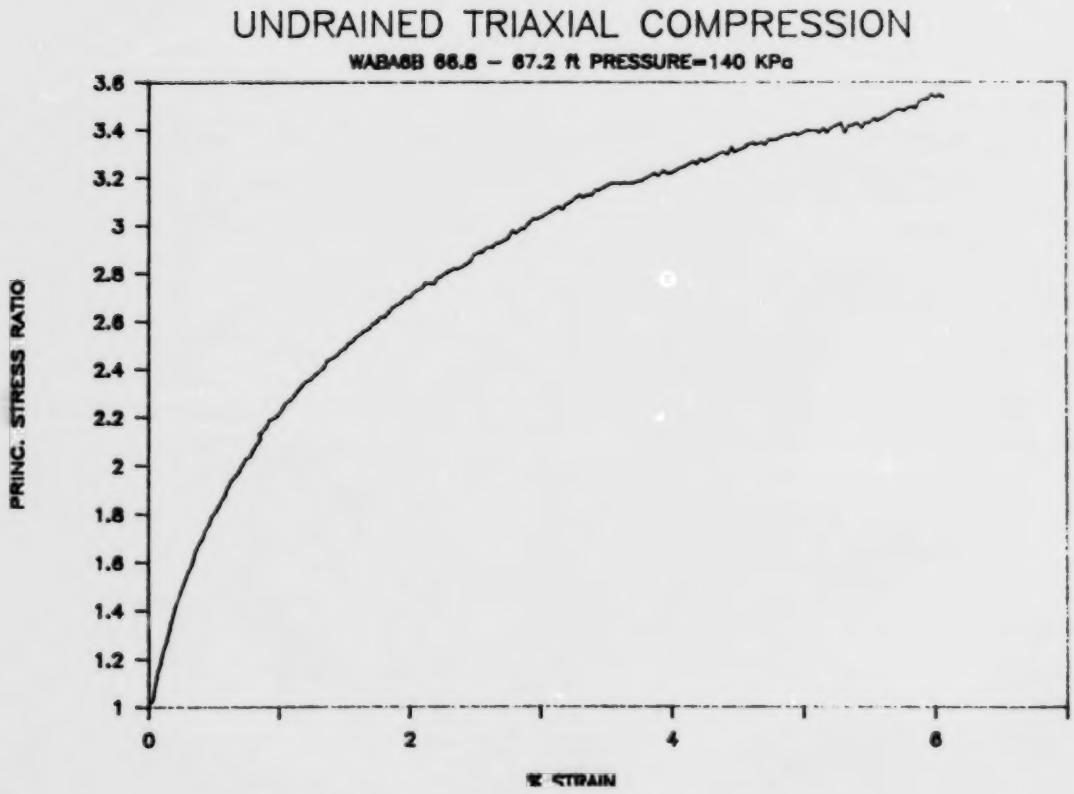
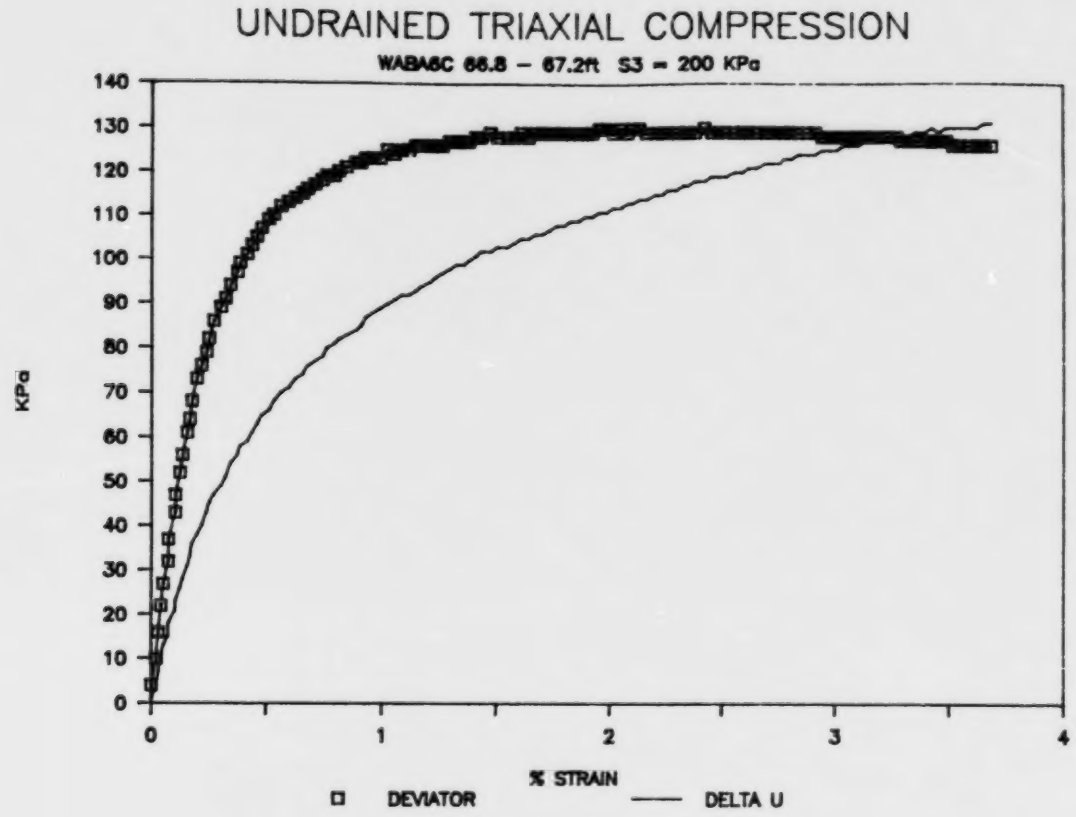
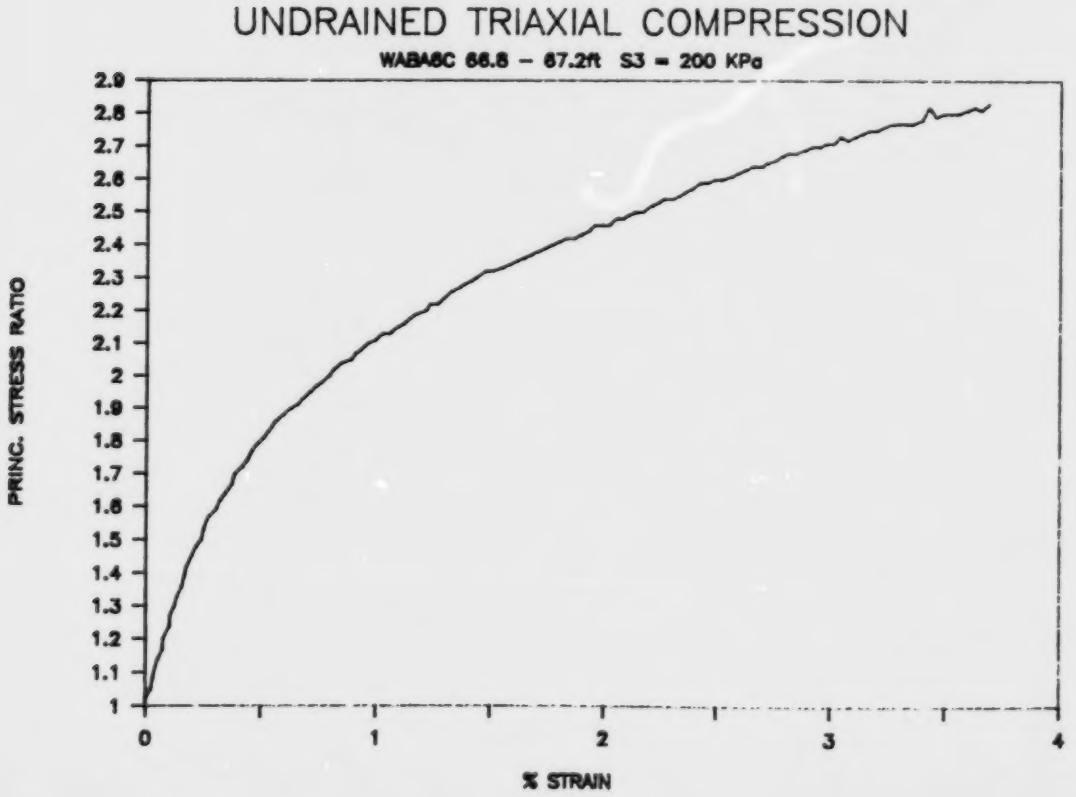

UNDRAINED TRIAXIAL COMPRESSION
WABA6B 66.8 - 67.2 ft PRESSURE=140 KPa
PRINC. STRESS RATIO
3.6
3.4
3.2
3
2.8
2.6
2.4
2.2
2
1.8
1.6
1.4
1.2
1
0
2
4
6
% STRAIN

FIGURE A11

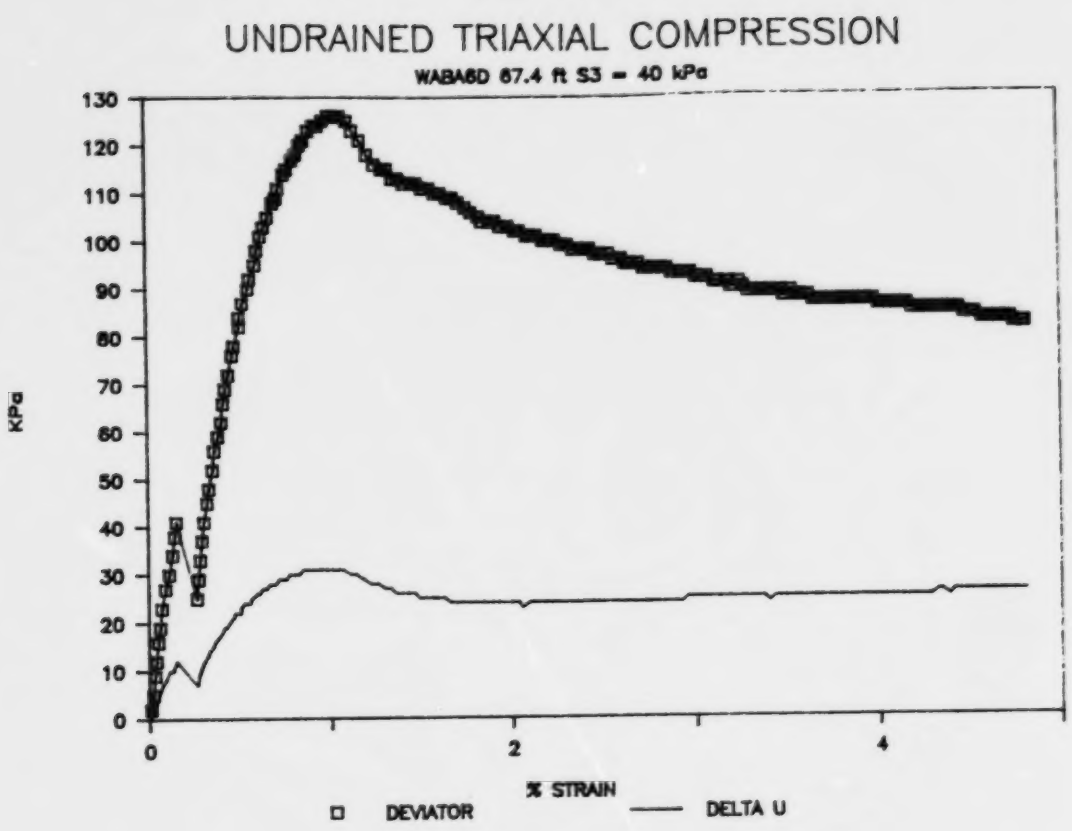


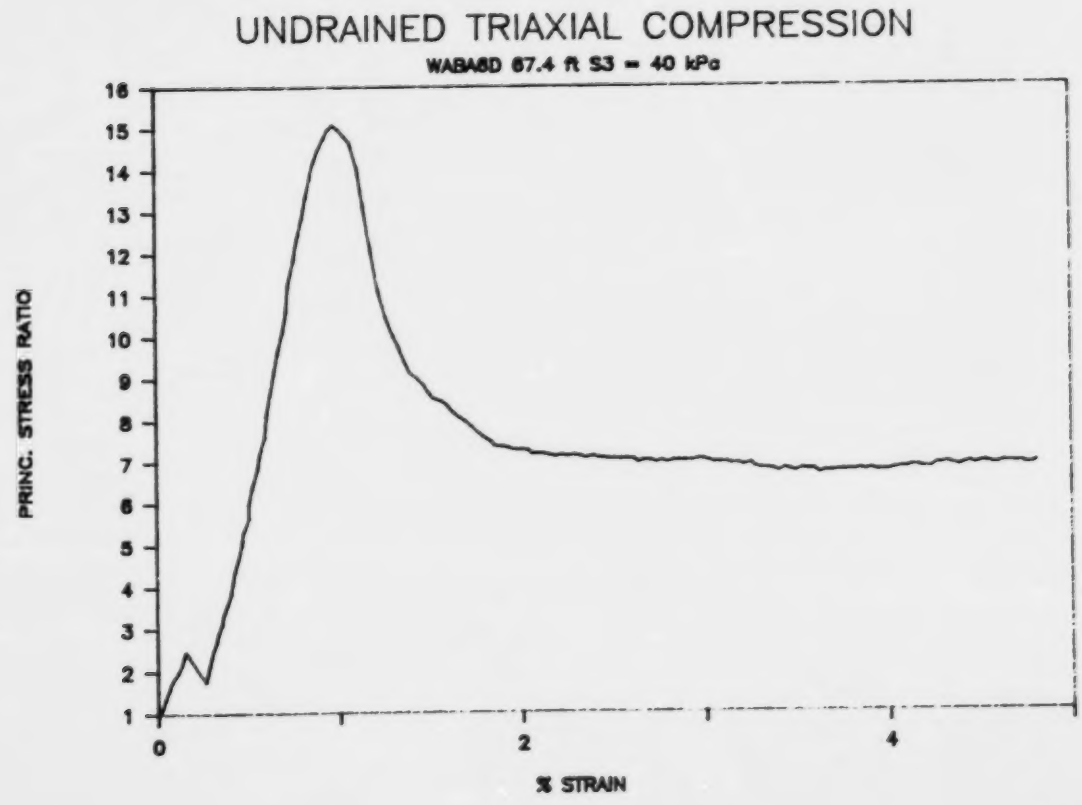


176213 RD

87-329

FIGURE A12

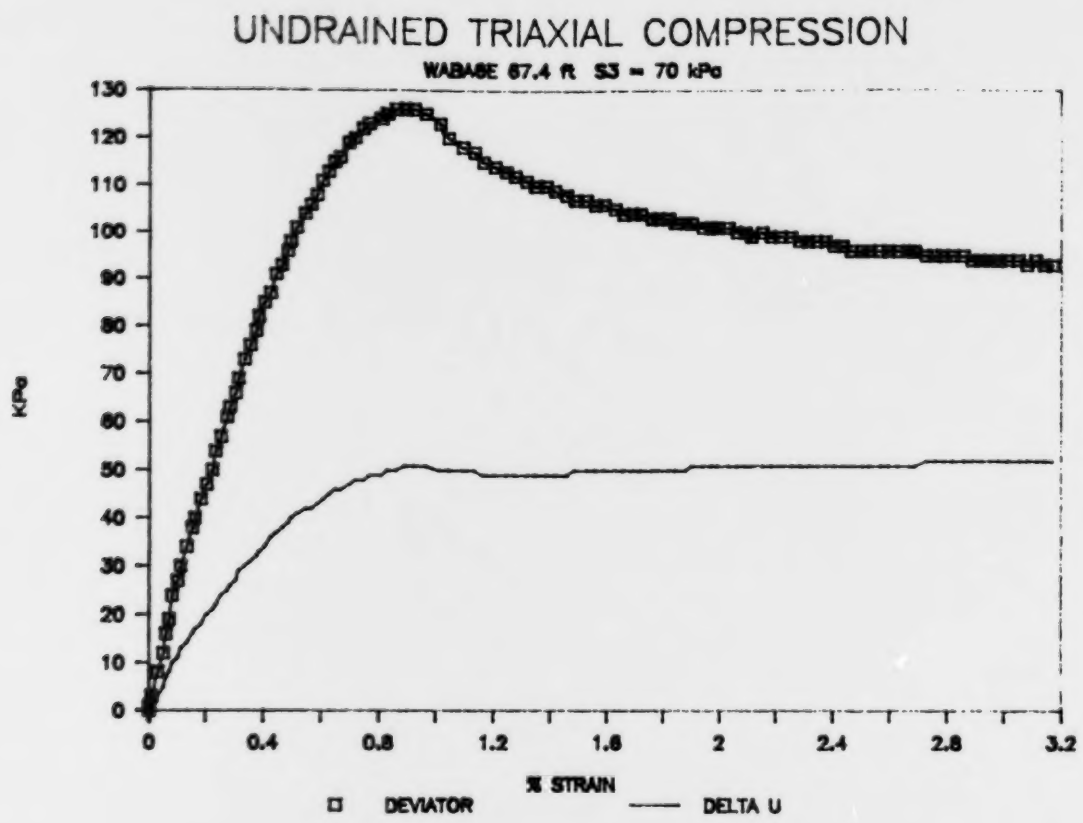




176214 RD

87-329

FIGURE A13


UNDRAINED TRIAXIAL COMPRESSION
WABASE 67.4 ft S3 = 70 kPa
KPa
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0
0.4
0.8
1.2
1.6
2
2.4
2.8
3.2
% STRAIN
DEVIATOR
DELTA U

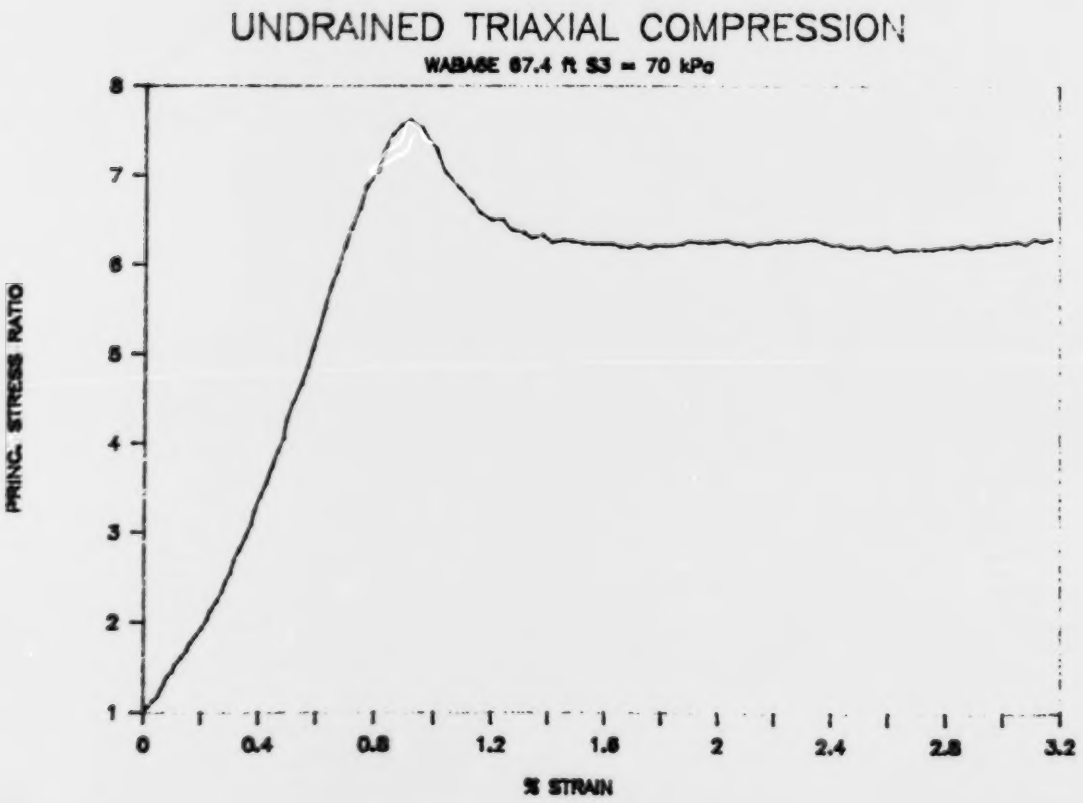
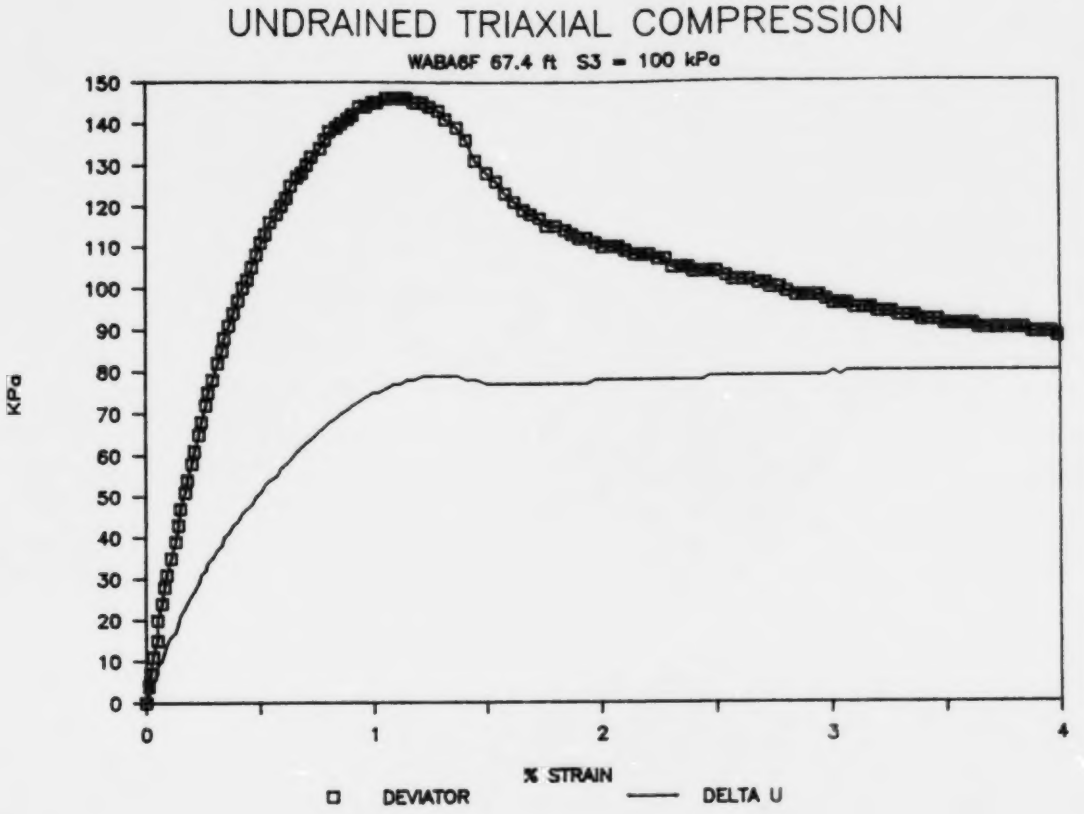
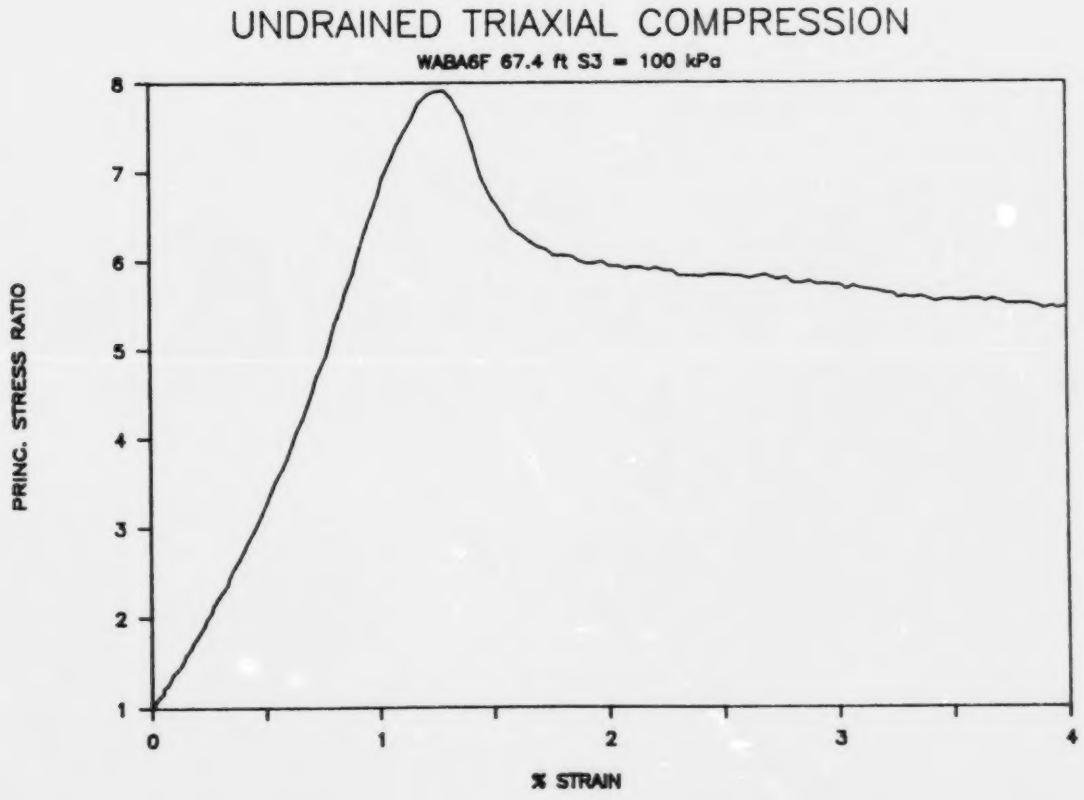

UNDRAINED TRIAXIAL COMPRESSION
WABASE 67.4 ft S3 = 70 kPa
PRINC. STRESS RATIO
8
7
6
5
4
3
2
1
0
0.4
0.8
1.2
1.6
2
2.4
2.8
3.2
% STRAIN

FIGURE A14


UNDRAINED TRIAXIAL COMPRESSION
WABA6F 67.4 ft S3 = 100 kPa
KPa
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0
1
2
3
4
% STRAIN
DEVIATOR
DELTA U

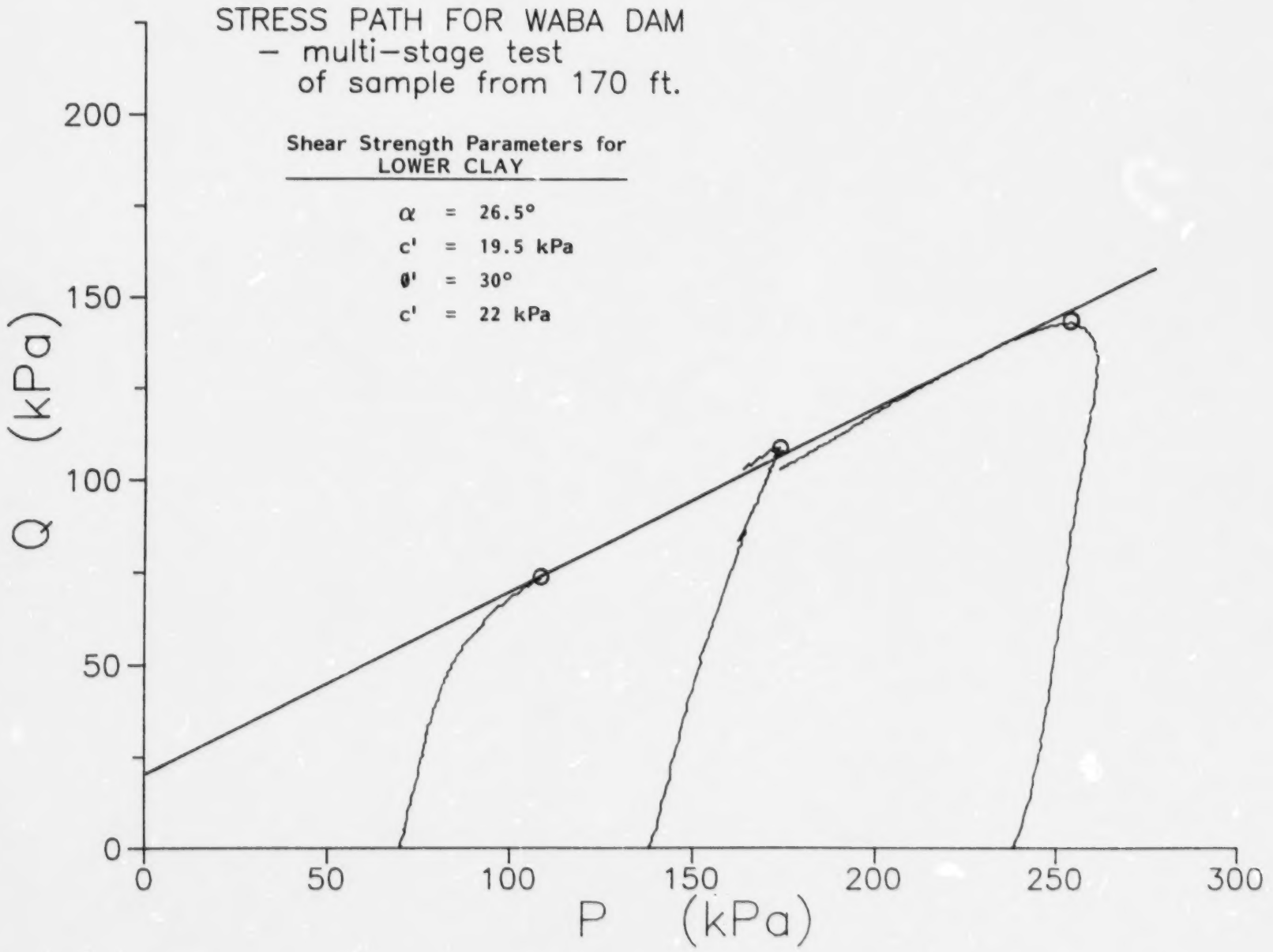

UNDRAINED TRIAXIAL COMPRESSION
WABA6F 67.4 ft S3 = 100 kPa
PRINC. STRESS RATIO
8
7
6
5
4
3
2
1
0
1
2
3
4
% STRAIN

CIU TESTS - LOWER CLAY

SAMPLE NO.	SAMPLE DEPTH (ft)	CONSOLIDATION PRESSURE (kPa)	BULK DENSITY (kg/m^3)	WATER CONTENT (%)
22	170	70 - 140 - 240*	1759	48.3

*MULTI-STAGE TEST

FIGURE A15

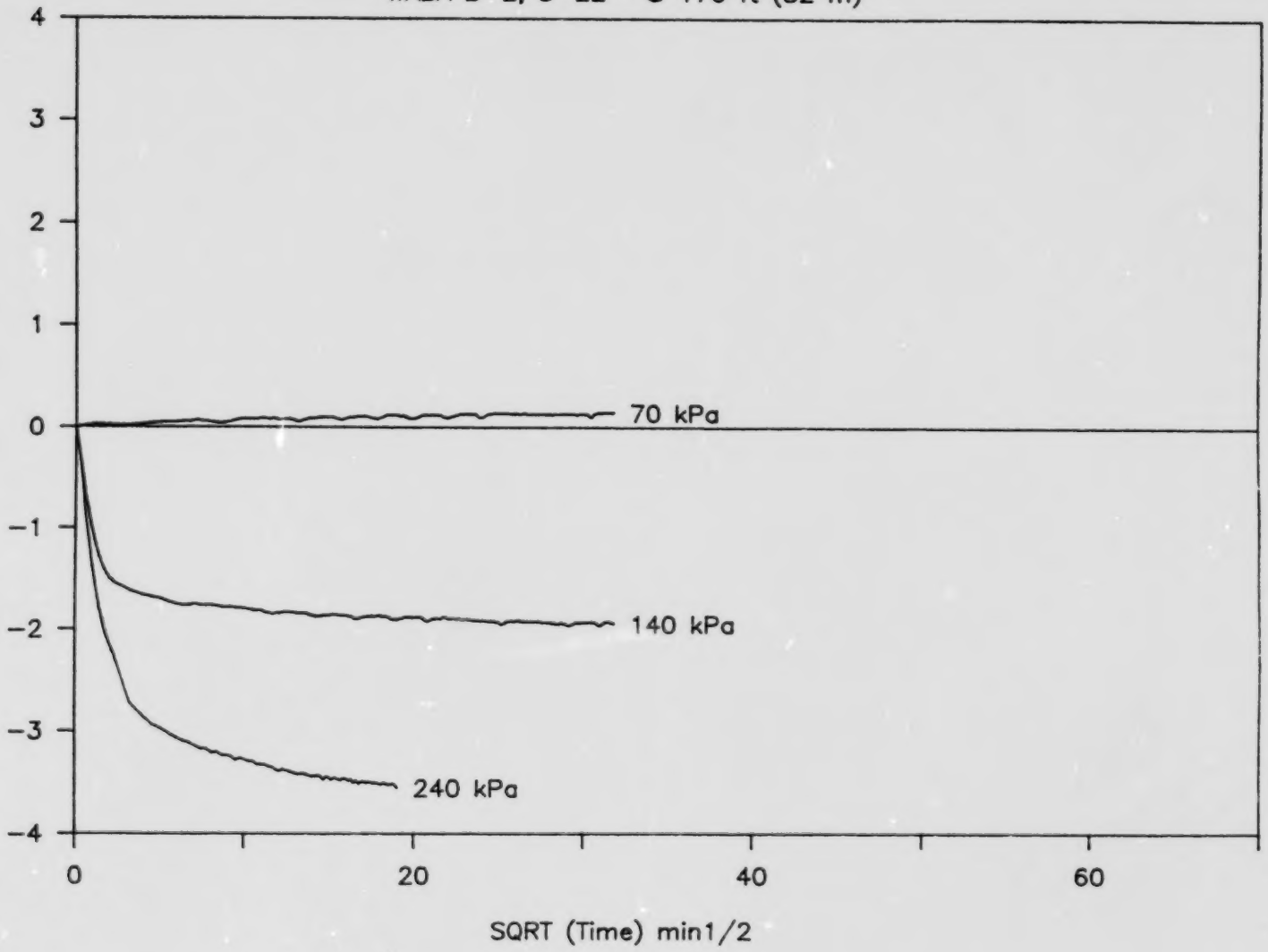


176241 RD
87-329

FIGURE A16

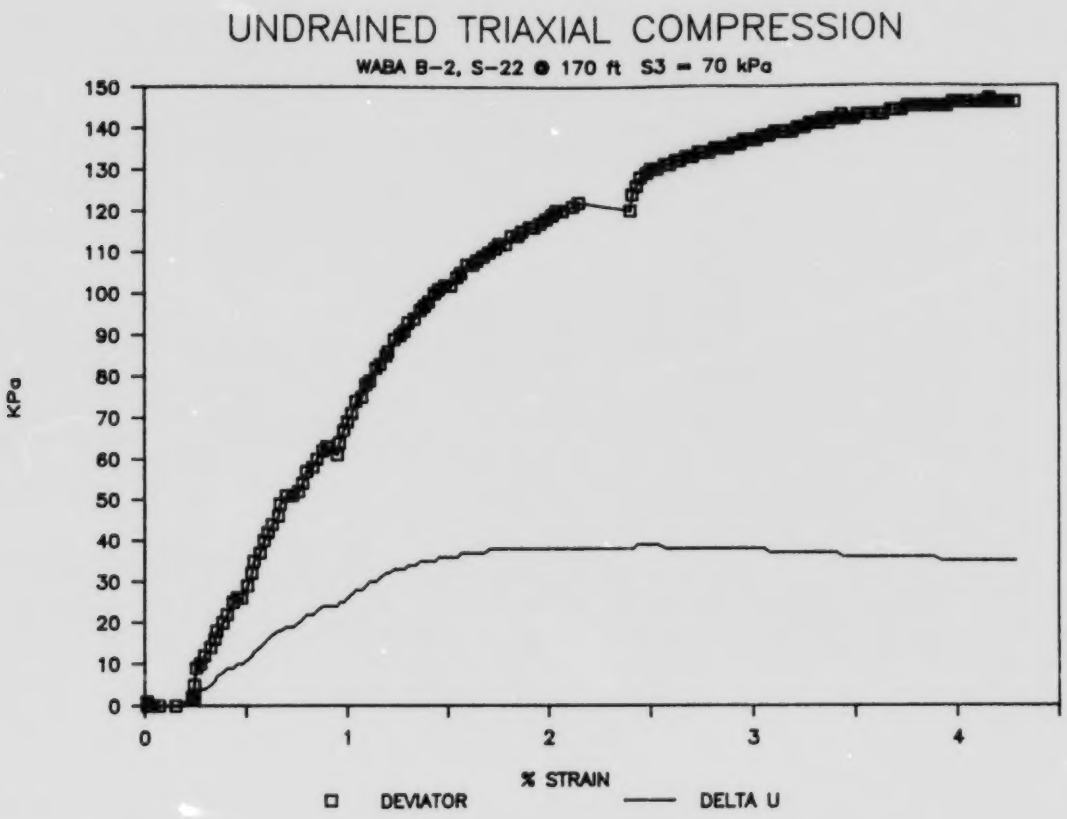
TRIAXIAL CONSOLIDATION

WABA B-2, S-22 @ 170 ft (52 m)



176217 RD
87-329

FIGURE A17


UNDRAINED TRIAXIAL COMPRESSION
WABA B-2, S-22 @ 170 ft S3 = 70 kPa
KPa
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0
1
2
3
4
% STRAIN
DEVIATOR
DELTA U

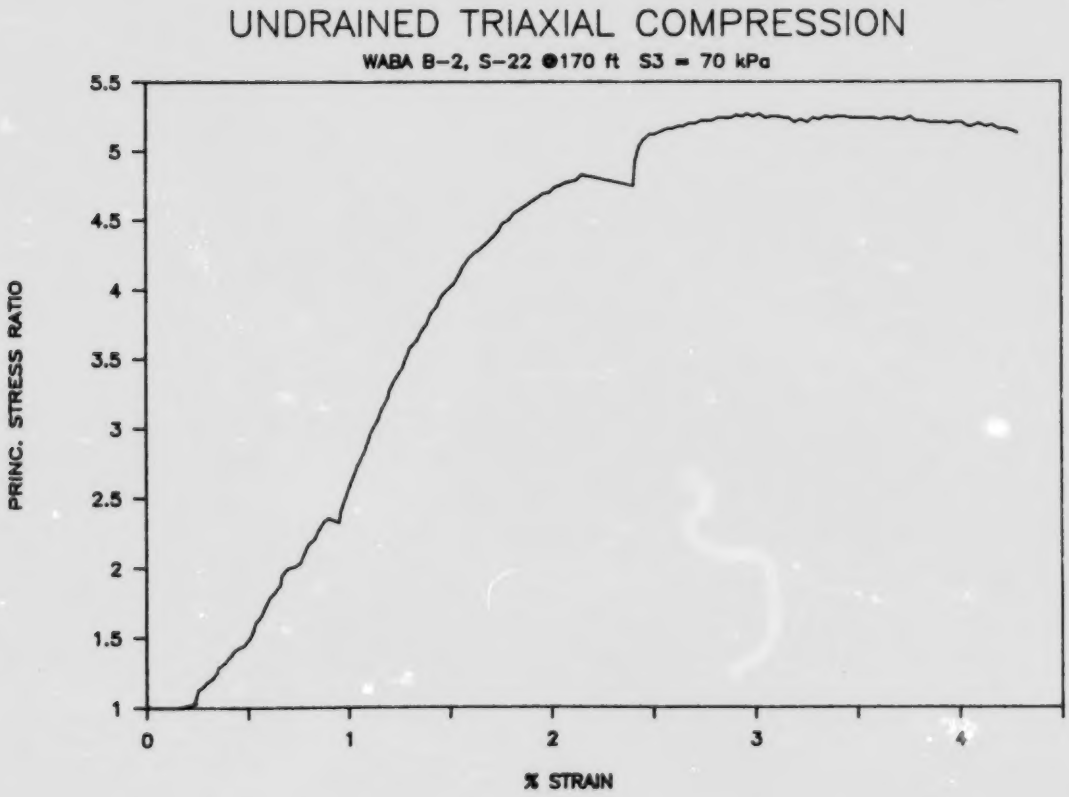
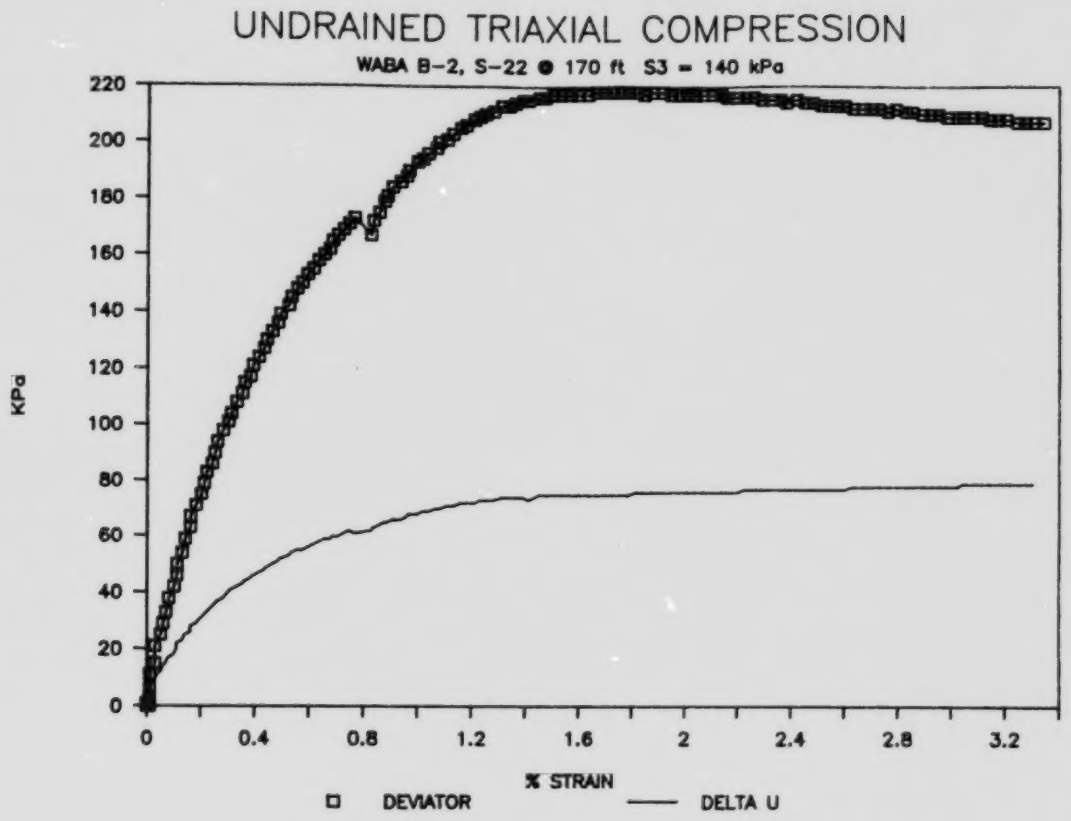
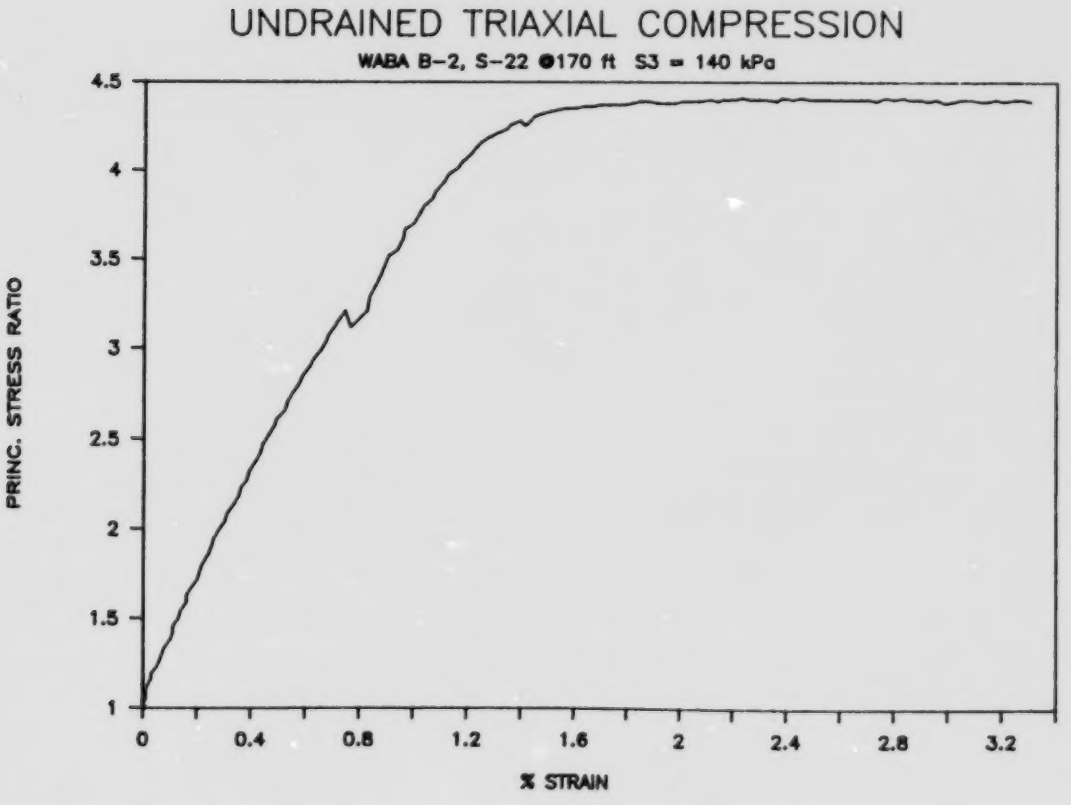

UNDRAINED TRIAXIAL COMPRESSION
WABA B-2, S-22 @170 ft S3 = 70 kPa
PRINC. STRESS RATIO
5.5
5
4.5
4
3.5
3
2.5
2
1.5
1
0
1
2
3
4
% STRAIN

FIGURE A18



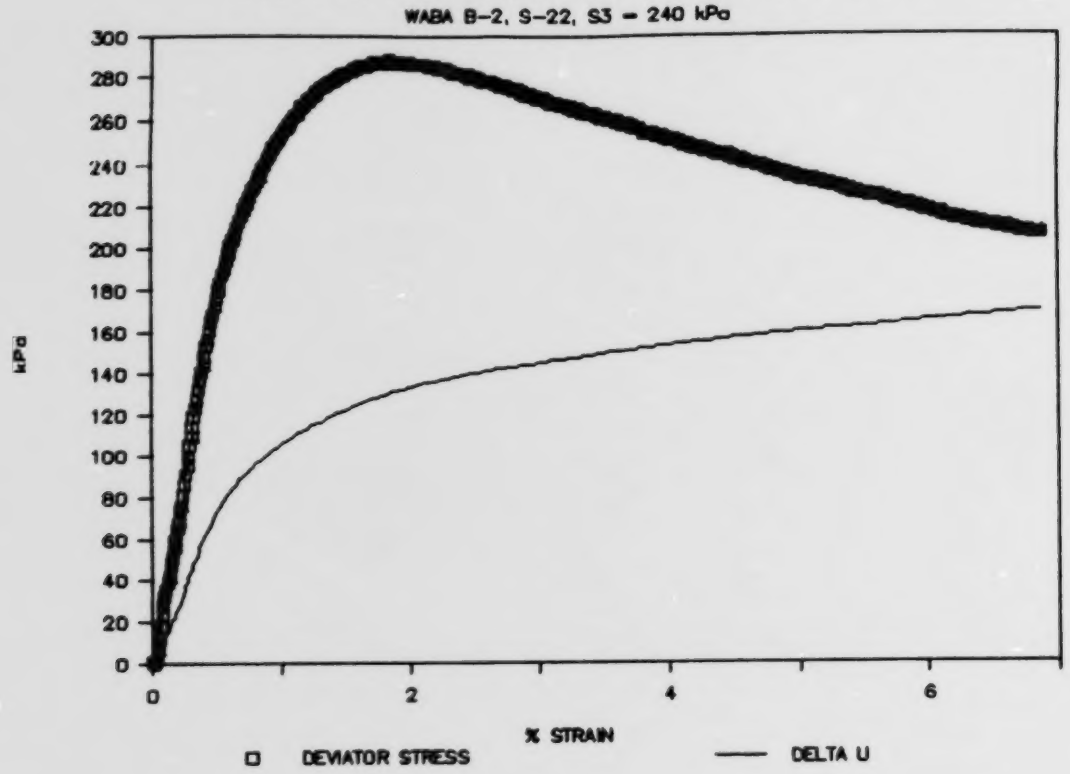


176220 RD
87-329

FIGURE A19

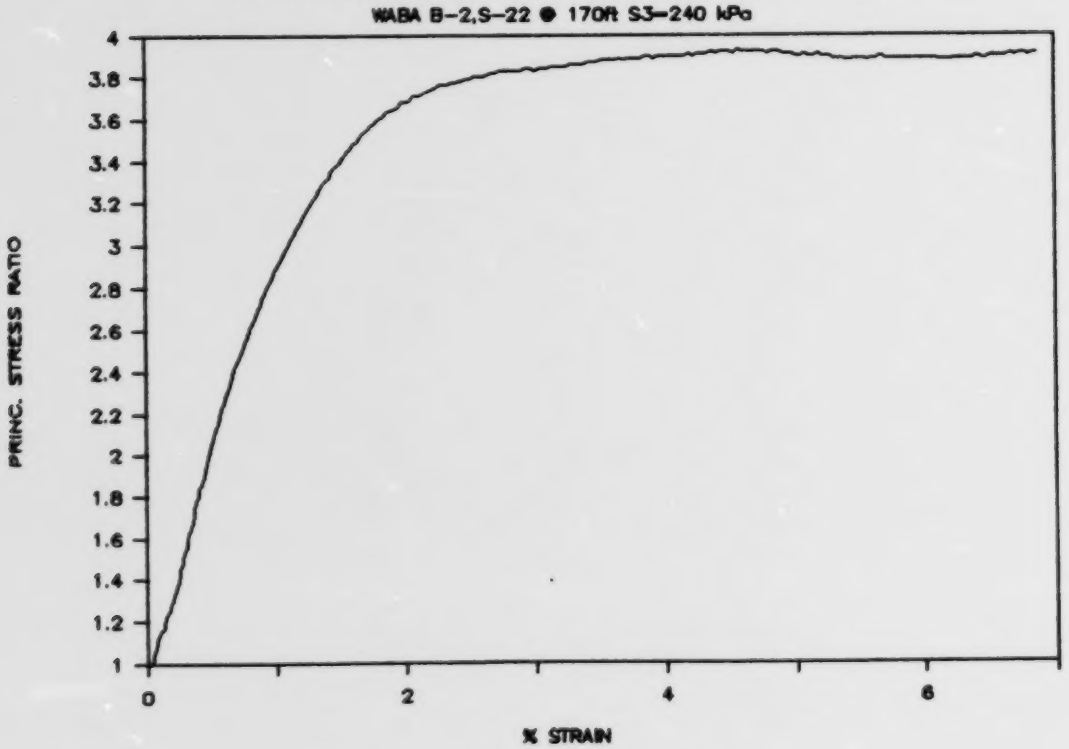
UNDRAINED TRIAXIAL COMPRESSION

WABA B-2, S-22, S3 = 240 kPa



UNDRAINED TRIAXIAL COMPRESSION

WABA B-2,S-22 @ 170ft S3=240 kPa



APPENDIX B

WABA DAM - PHASE II

CONSOLIDATION TESTS

TABLE B1

WABA DAM - CONSOLIDATION TEST RESULTS

Sample Depth (ft)	Sample No	Preconsolidation Pressure (P_c') (psf)	Coefficient of Consolidation* ($inch^2/min$)	Initial Water Content (%)
59	4	2,300	0.0071	74.7
91	11	3,000	0.0093	72.0
121	16	4,000	0.0025	59.0
136	18	4,800	0.0101	45.4
161	21	6,000	0.0454	36.3
182	23	7,500	0.1134	43.0
192	24	8,000	0.1935	38.2
201	25	16,000	0.0182	48.1

*load nearest P_c'

TABLE B2

WABA DAM - COEFFICIENT OF CONSOLIDATION

Pressure Increment (psf)	Coefficient of Consolidation (in^2/min) Sample Depth (ft)			
	59'	91'	121'	136'
1000	0.0032	0.0164	0.0054	0.106
2000	0.0071	0.0285	0.0211	0.0046
4000	0.0023	0.0093	0.0025	0.0101
8000	0.0005	0.0004	0.0059	0.0039
	161'	182'	192'	201'
1000	0.0499	0.1216	0.0990	0.1991
2000	0.0669	0.1522	0.1213	0.0569
4000	0.1161	0.0958	0.0802	0.0945
8000	0.454	0.1134	0.1935	0.1438

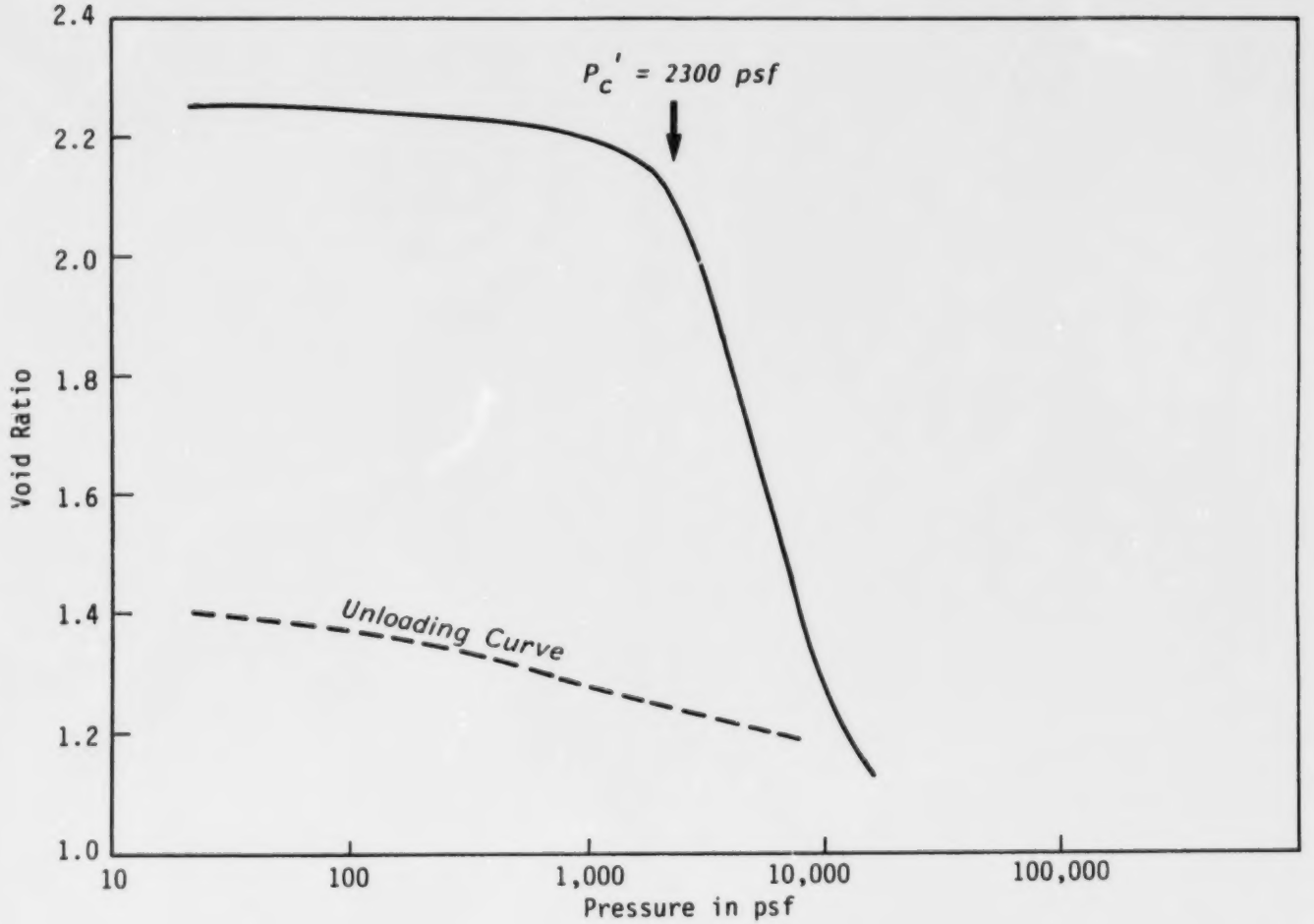


FIGURE B1

CONSOLIDATION TEST
WABA DAM
BH.2 S-4 59'

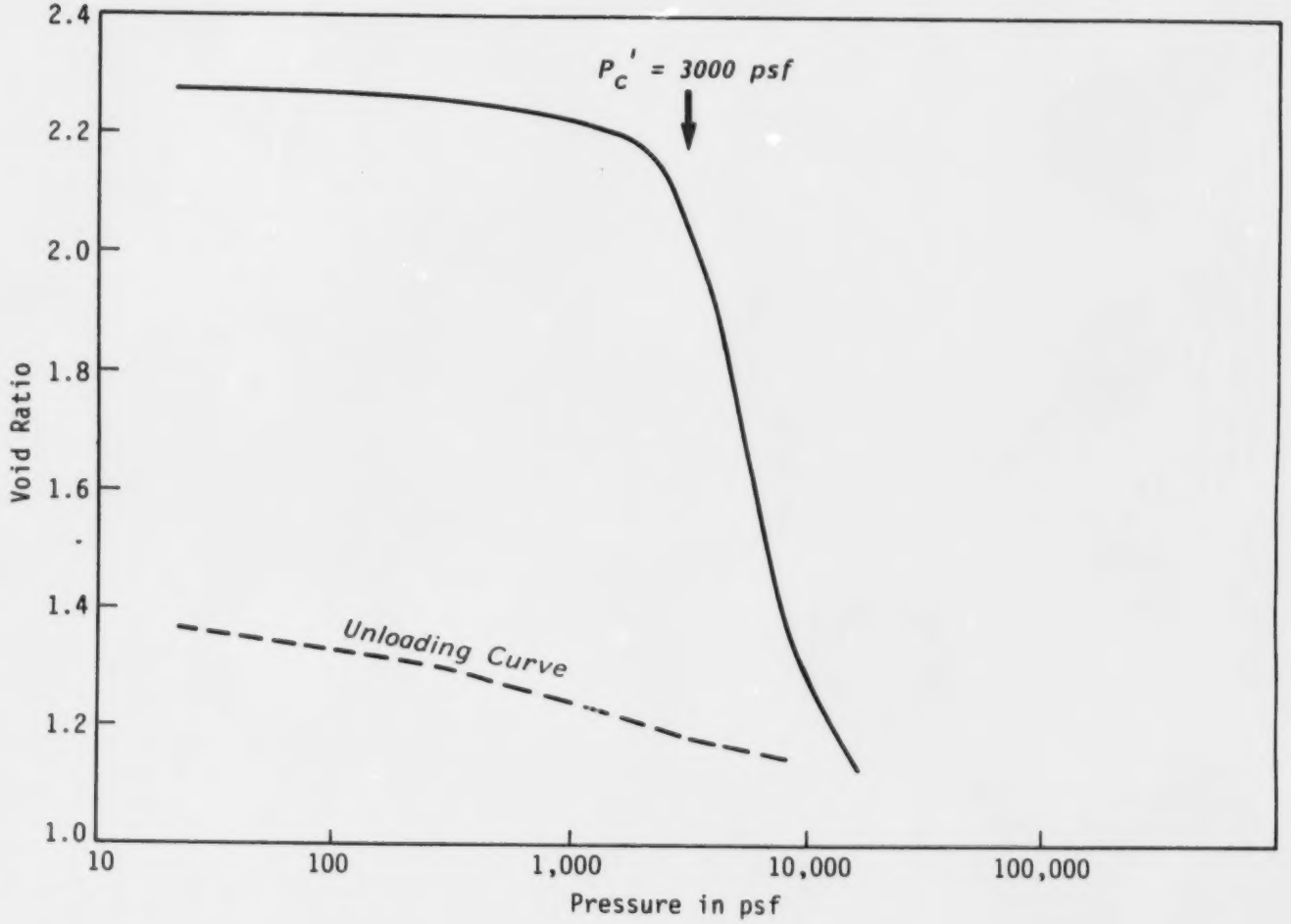


FIGURE B2

CONSOLIDATION TEST
WABA DAM
BH.2 S-11 91'

87-329 176169 RD

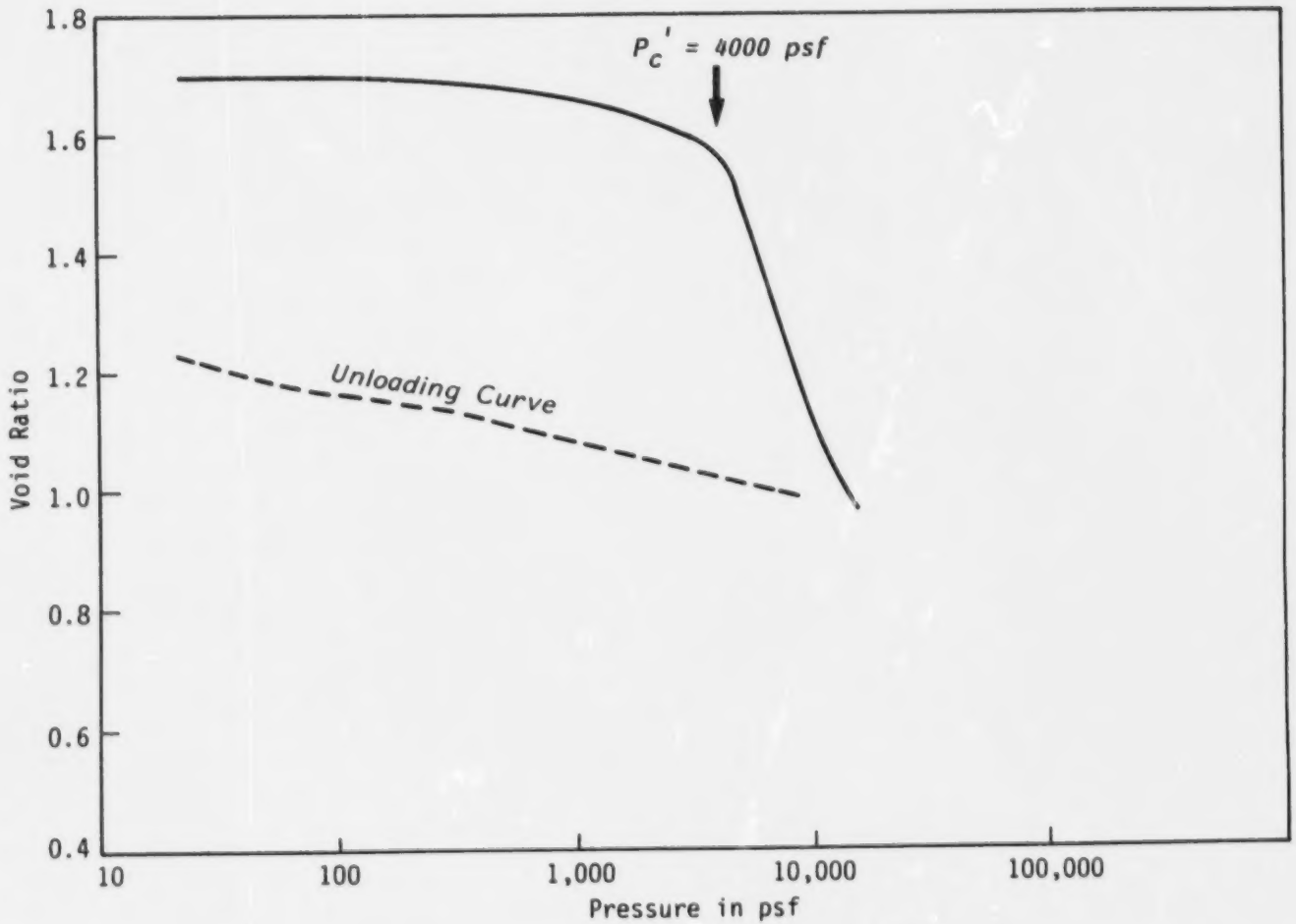


FIGURE B3

CONSOLIDATION TEST
WABA DAM
BH.2 S-16 121'

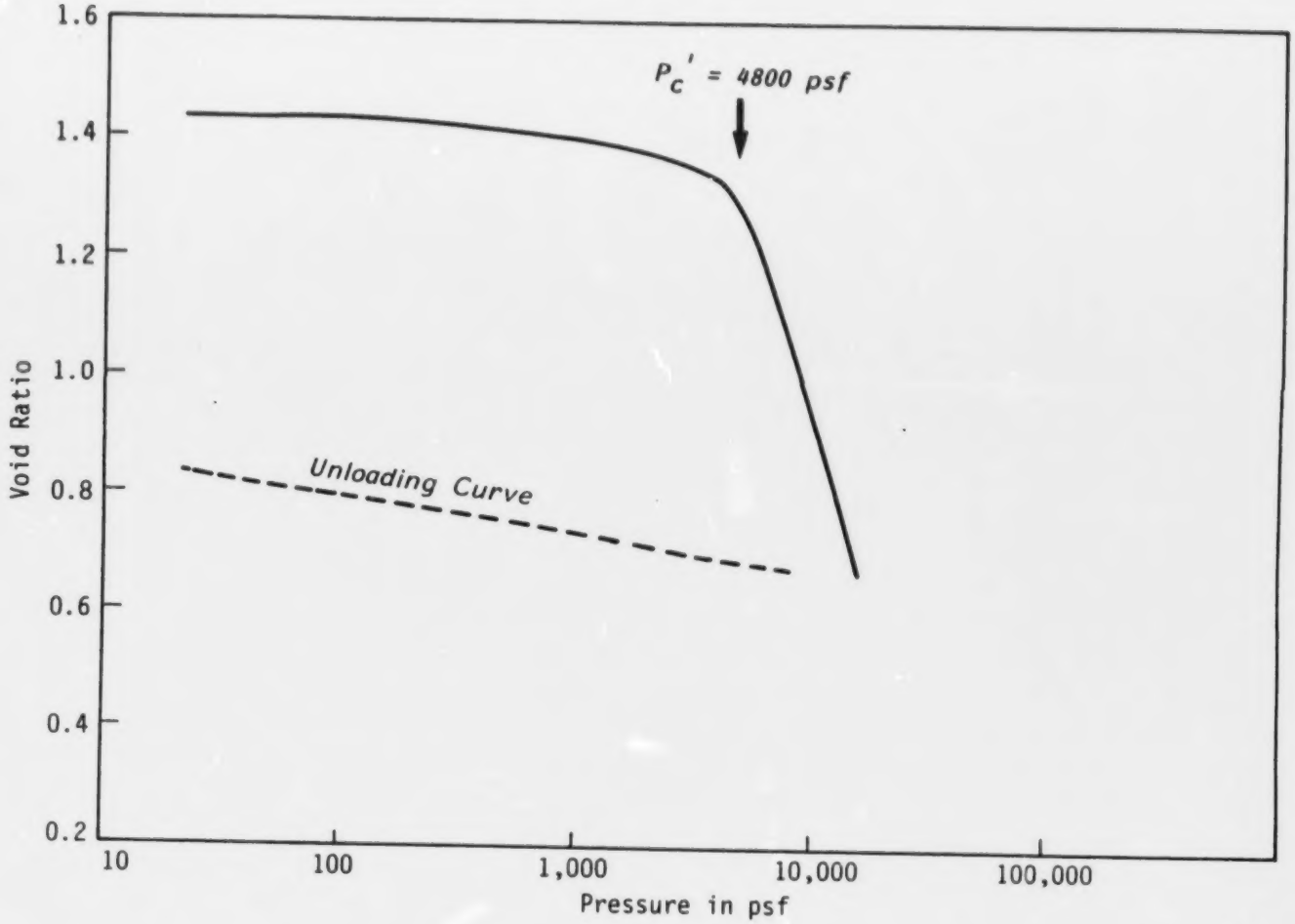


FIGURE B4

CONSOLIDATION TEST
WABA DAM
BH.2 S-18 136'

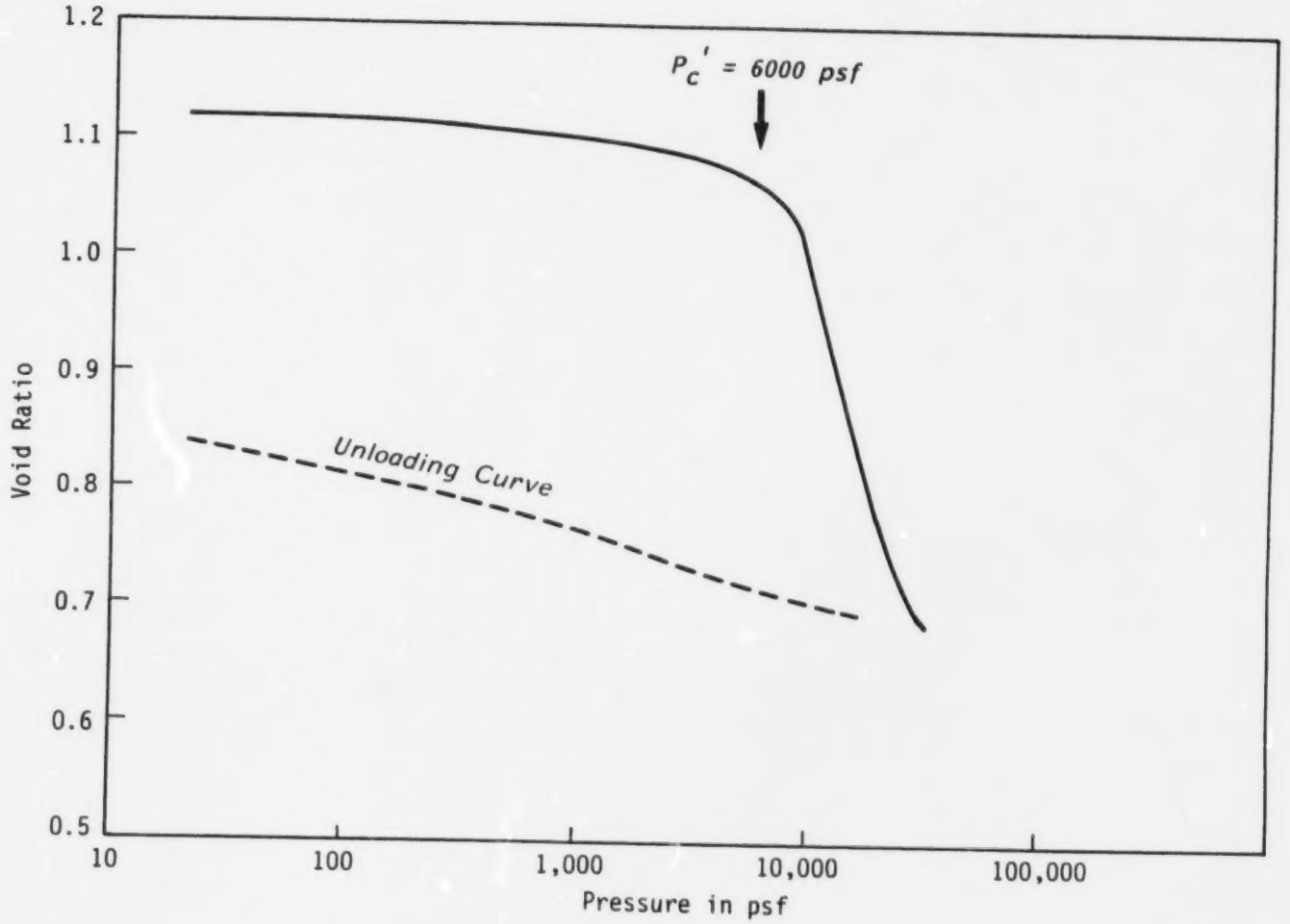


FIGURE B5

CONSOLIDATION TEST
WABA DAM
BH.2 S-21 161'

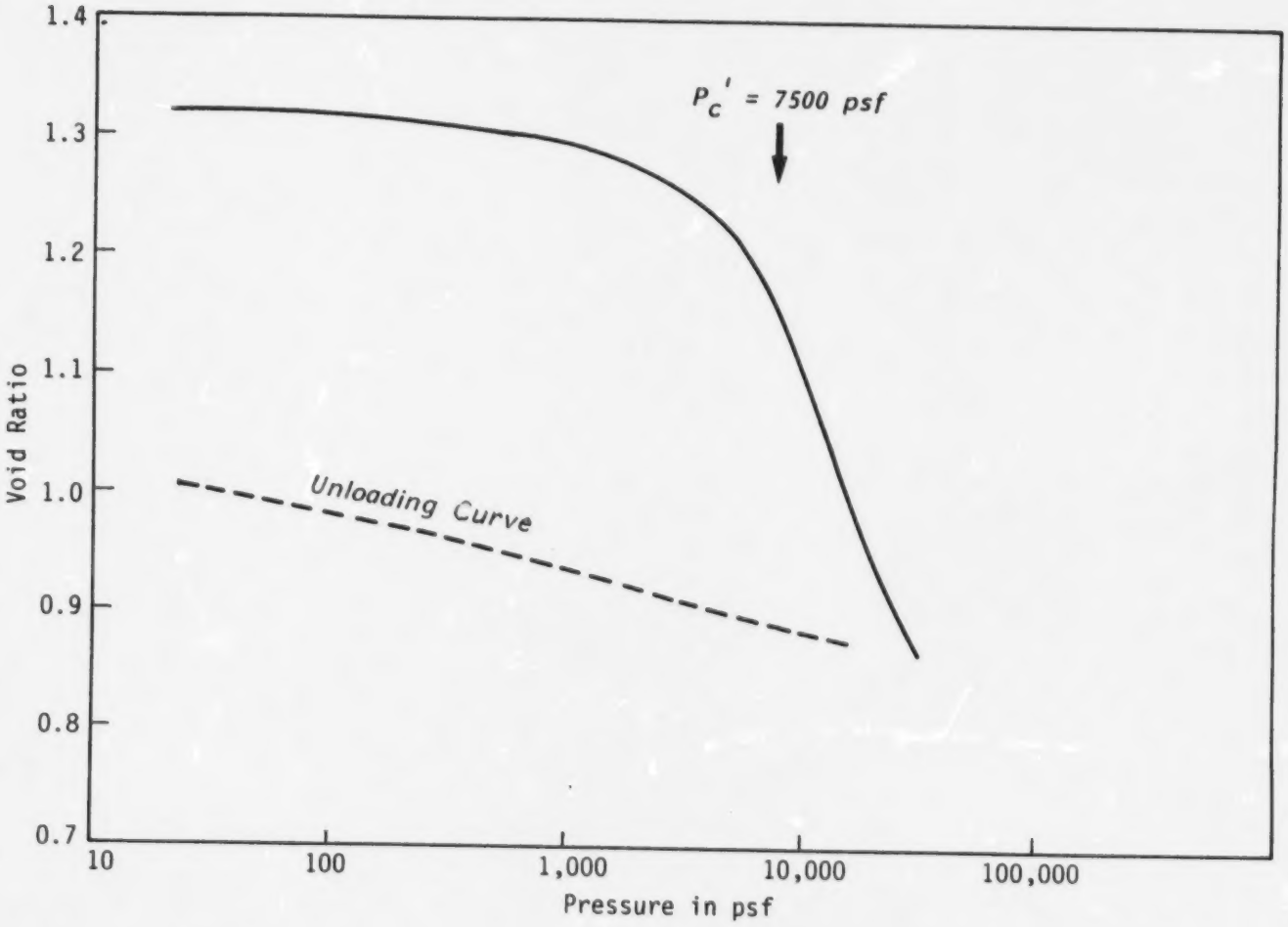


FIGURE B6

CONSOLIDATION TEST
WABA DAM
BH.2 S-23 181'

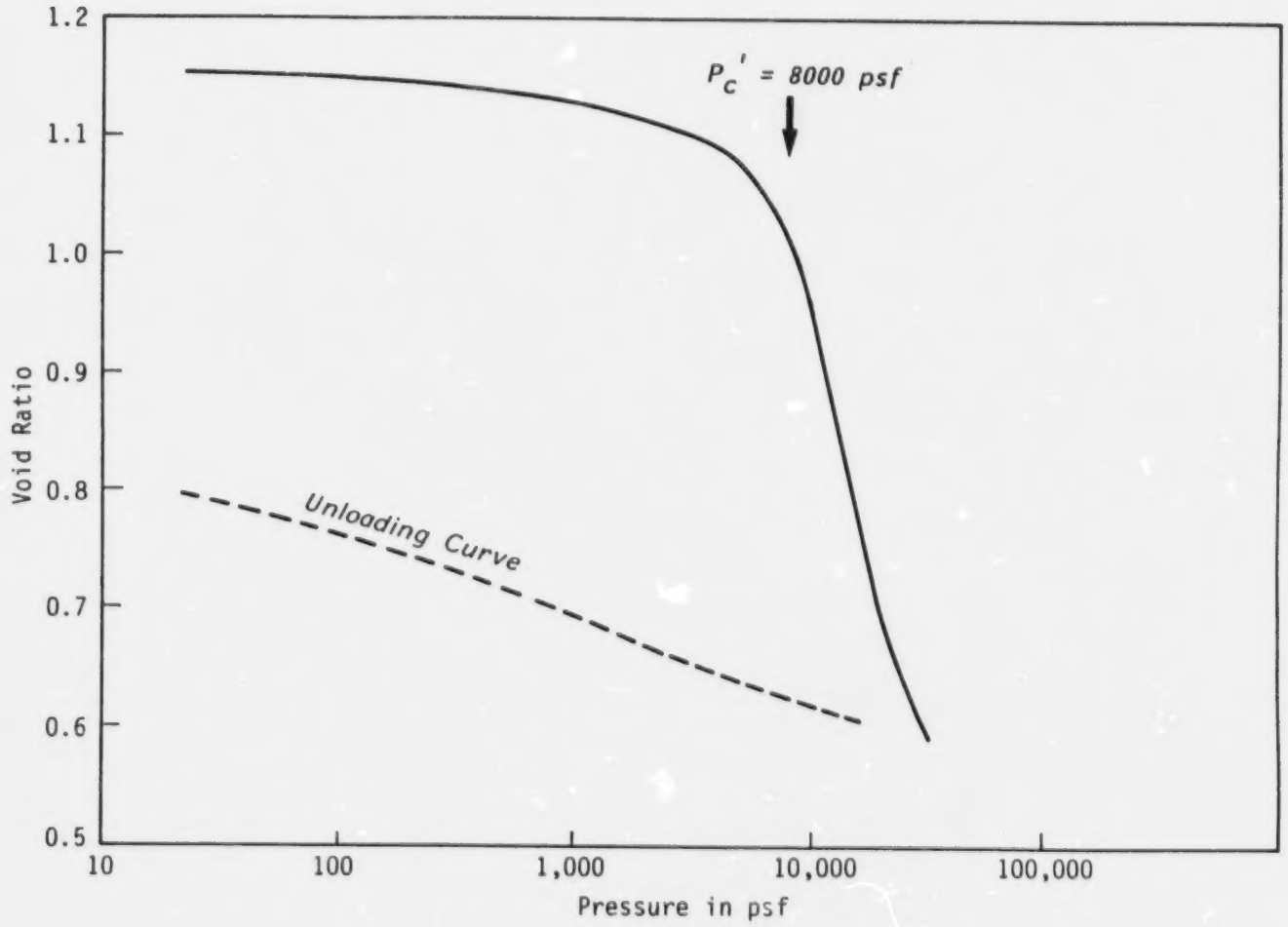


FIGURE B7

CONSOLIDATION TEST
WABA DAM
BH.2 S-24 191'

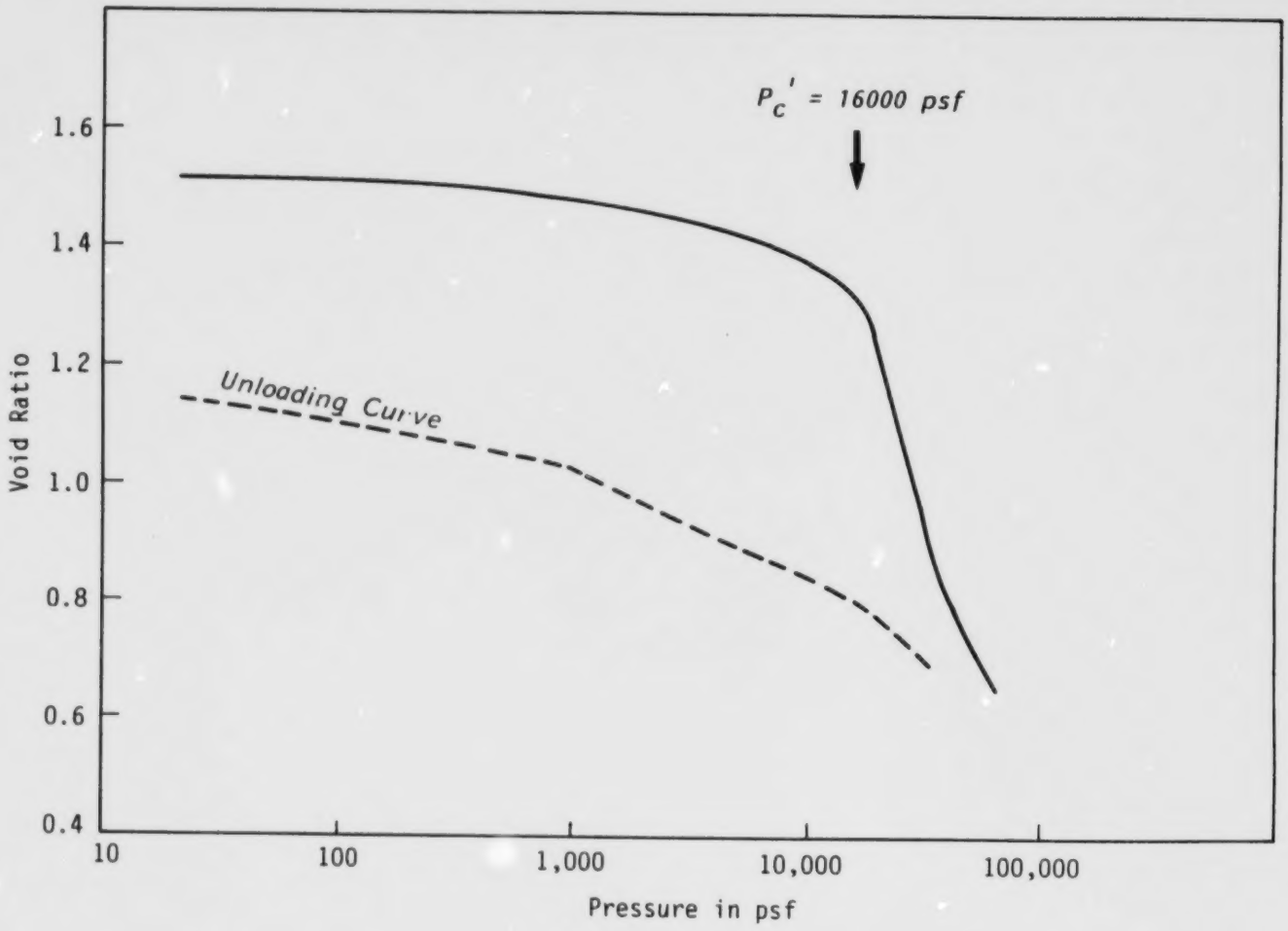


FIGURE B8

CONSOLIDATION TEST
WABA DAM
BH.2 S-25 201'

87-329 176175 RD

APPENDIX C

WABA DAM - PHASE II

RESONANT COLUMN TESTS

TABLE C1

WABA DAM - PHASE II
RESONANT COLUMN TEST SUMMARY

Test No	Sample Depth (ft)	Bulk Density (kg/m^3)	Water Content (%)	Maximum Shear Modulus (MPa)	Shear Strength (kPa)
1	59	1545	75.8	15.4*	30
2	91	1581	70.2	24.2*	65
3	111	1543	67.6	21.6*	40
4	131	1657	55.0	41.3*	47
5	140	1723	54.5	50.5*	55
6	246	1820	37.5	75.4R	191
7	161	1819	38.1	58.6R	165
8	180	1915	38.7	86.1*	163
9	201	1854	39.8	83.6*	200
10	231	1813	40.9	97.3	200
11	161	1828	41.0	73.5	165
12	246	1884	32.5	116.8	191
13	190	1798	38.5	80.1	158
14	171	1793	47.1	60.6	148
15	121	1621	64.9	29.5	61

R - poor test/redone
* - corrected value (correction factor = 2% to 12%)

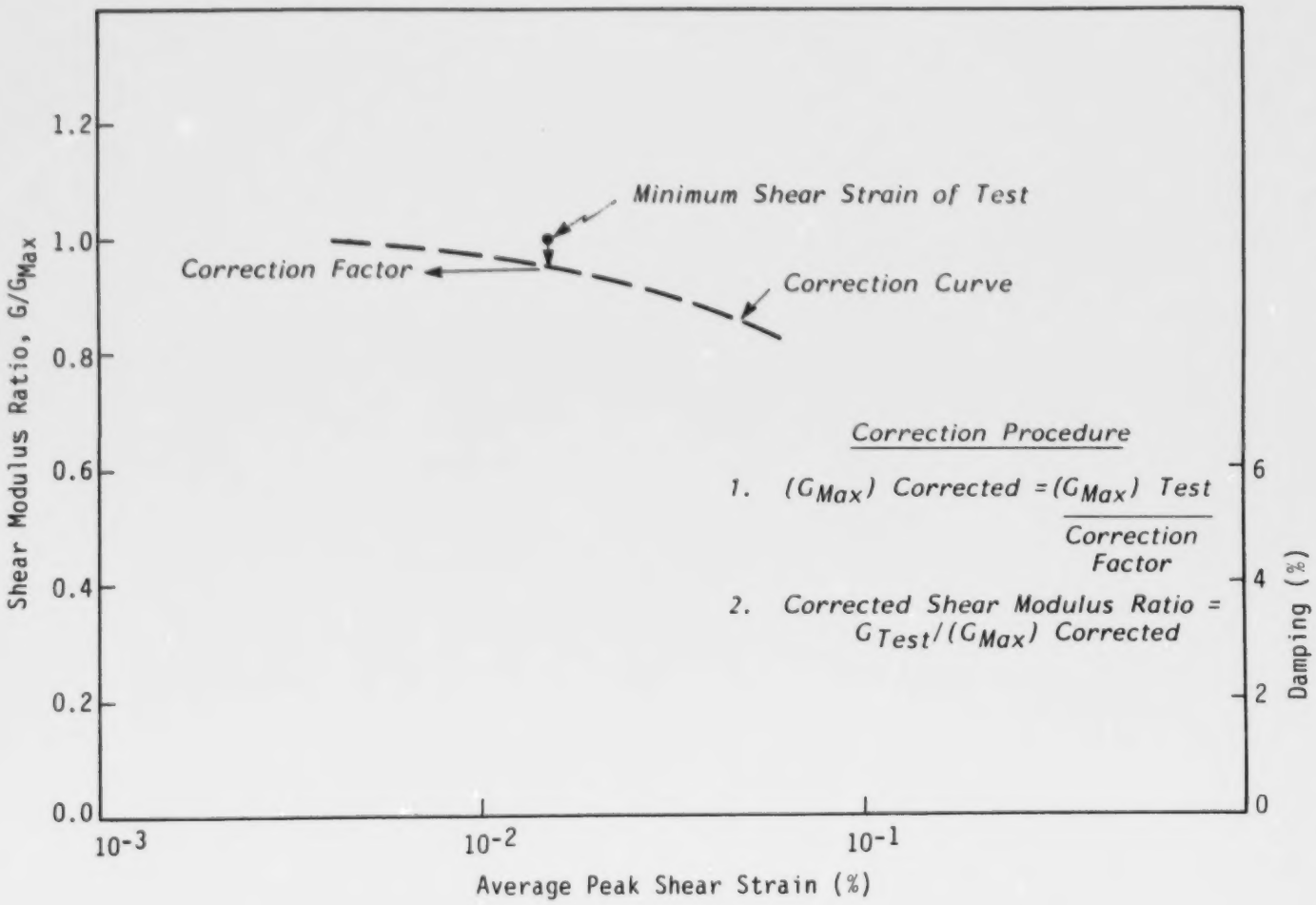


FIGURE C1

RESONANT COLUMN
CORRECTION FOR UPPER CLAY TESTS

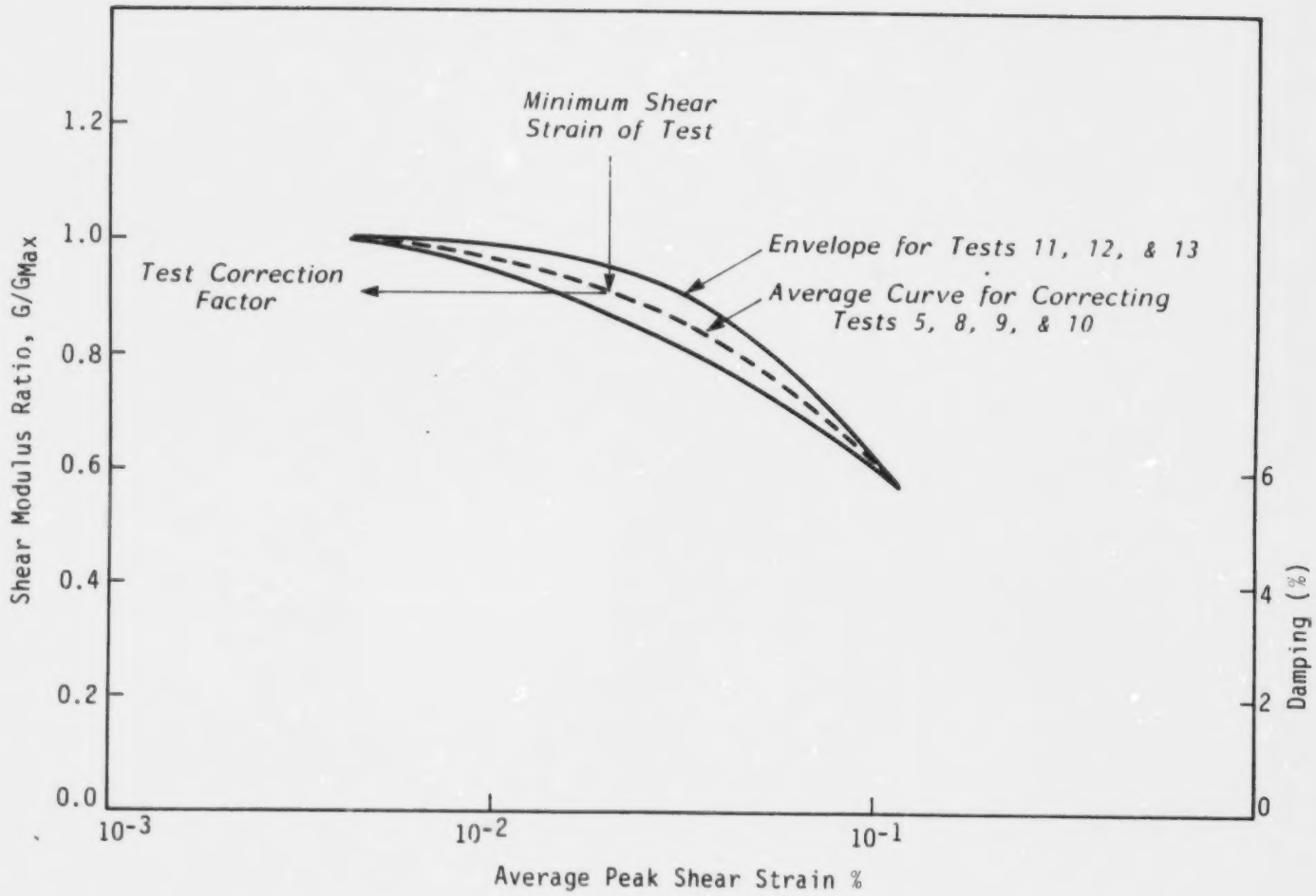


FIGURE C2

RESONANT COLUMN
CORRECTION METHOD FOR LOWER CLAY TESTS

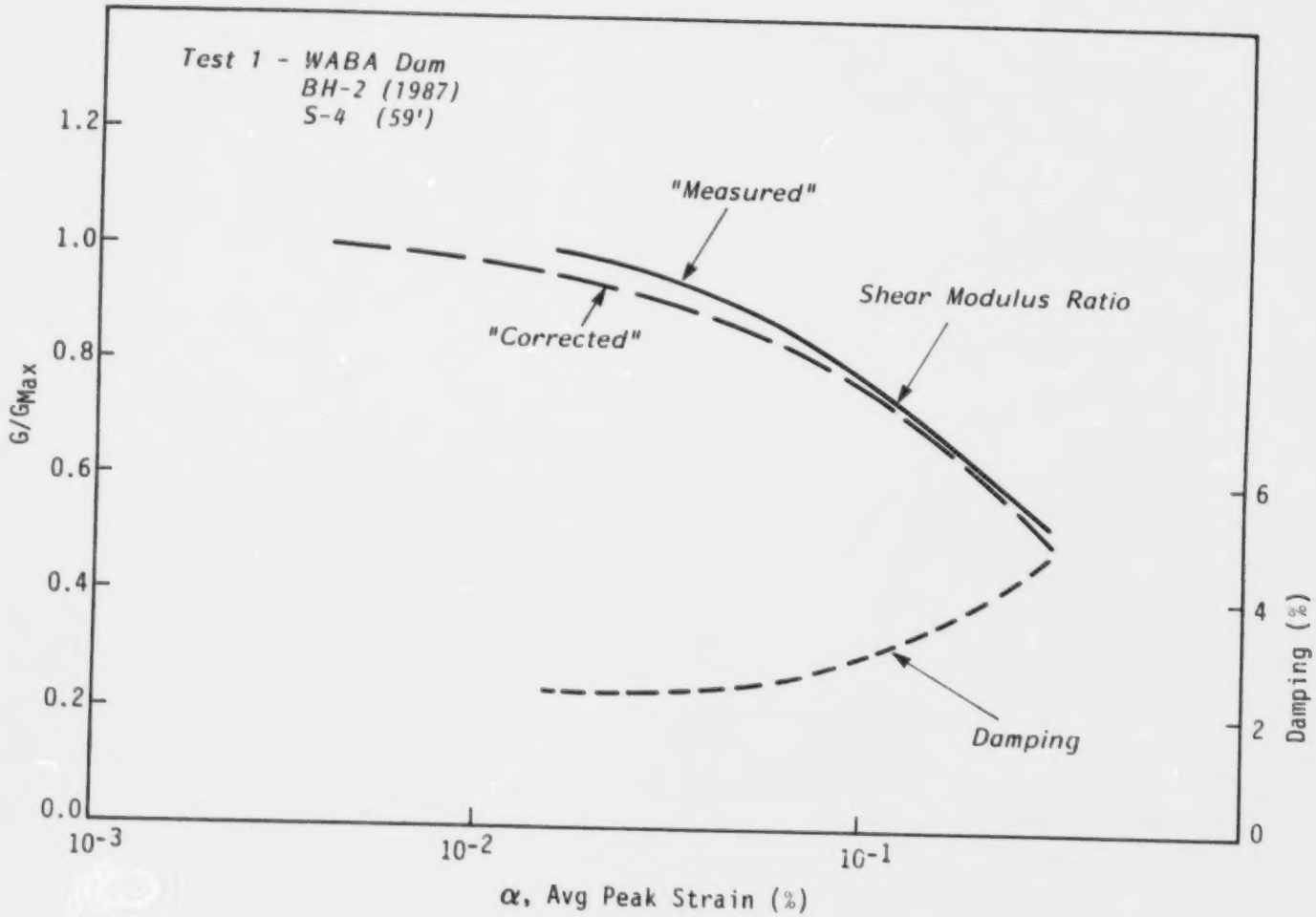

Test 1 - WABA Dam
BH-2 (1987)
S-4 (59')
"Measured"
"Corrected"
Shear Modulus Ratio
Damping
G/GMax
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
6
4
2
0
10-3
10-2
10-1

FIGURE C3

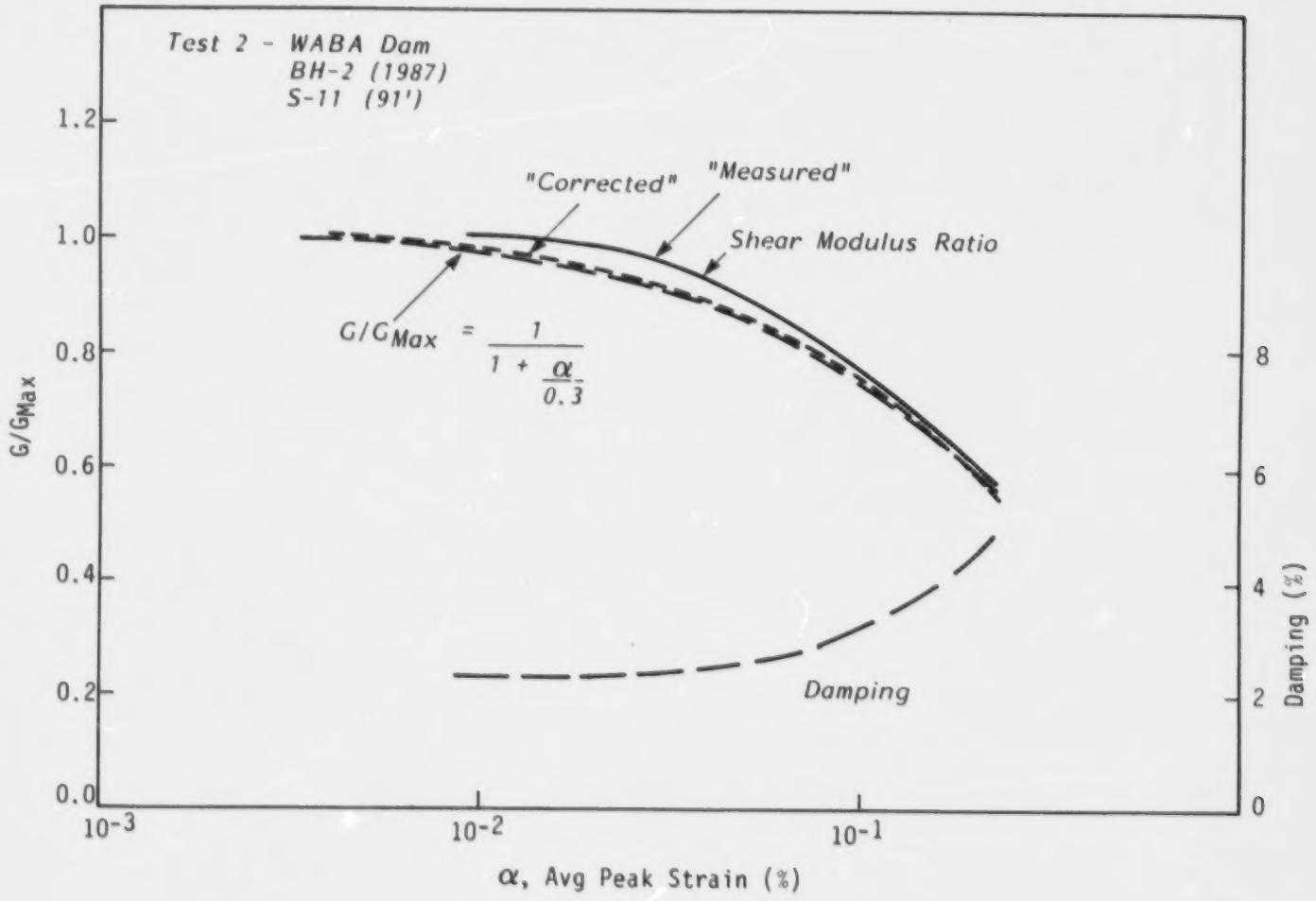

Test 2 - WABA Dam
BH-2 (1987)
S-11 (91')
"Corrected"
"Measured"
Shear Modulus Ratio
G/GMax = 1 / (1 + α/0.3)
Damping
G/GMax
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
8
6
4
2
0
10-3
10-2
10-1

FIGURE C4

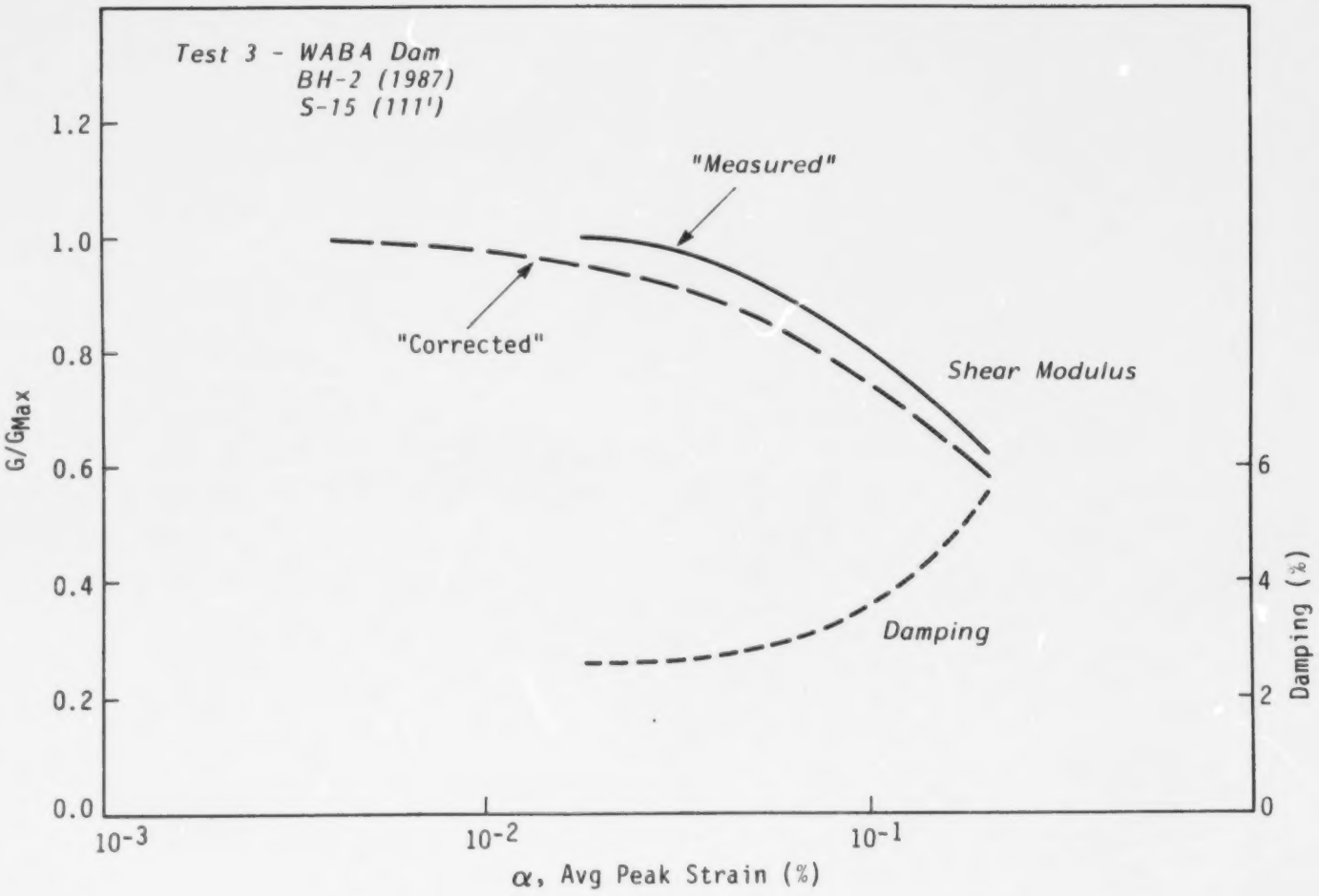

Test 3 - WABA Dam
BH-2 (1987)
S-15 (111')
"Measured"
"Corrected"
Shear Modulus
Damping
G/GMax
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
6
4
2
0
10-3
10-2
10-1

FIGURE C5

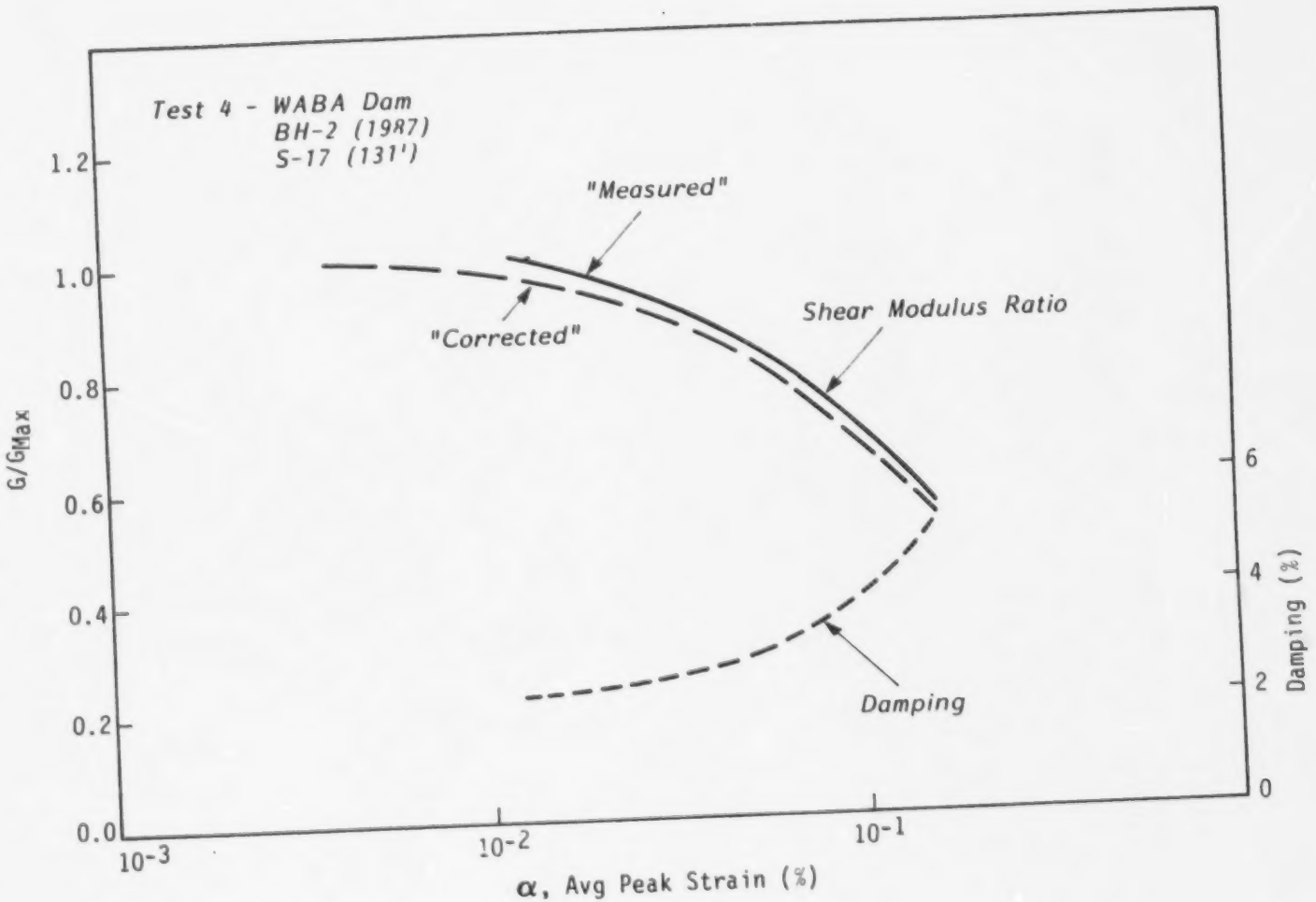

Test 4 - WABA Dam
BH-2 (1987)
S-17 (131')
"Measured"
"Corrected"
Shear Modulus Ratio
Damping
G/GMax
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
6
4
2
0
10-3
10-2
10-1

FIGURE C6

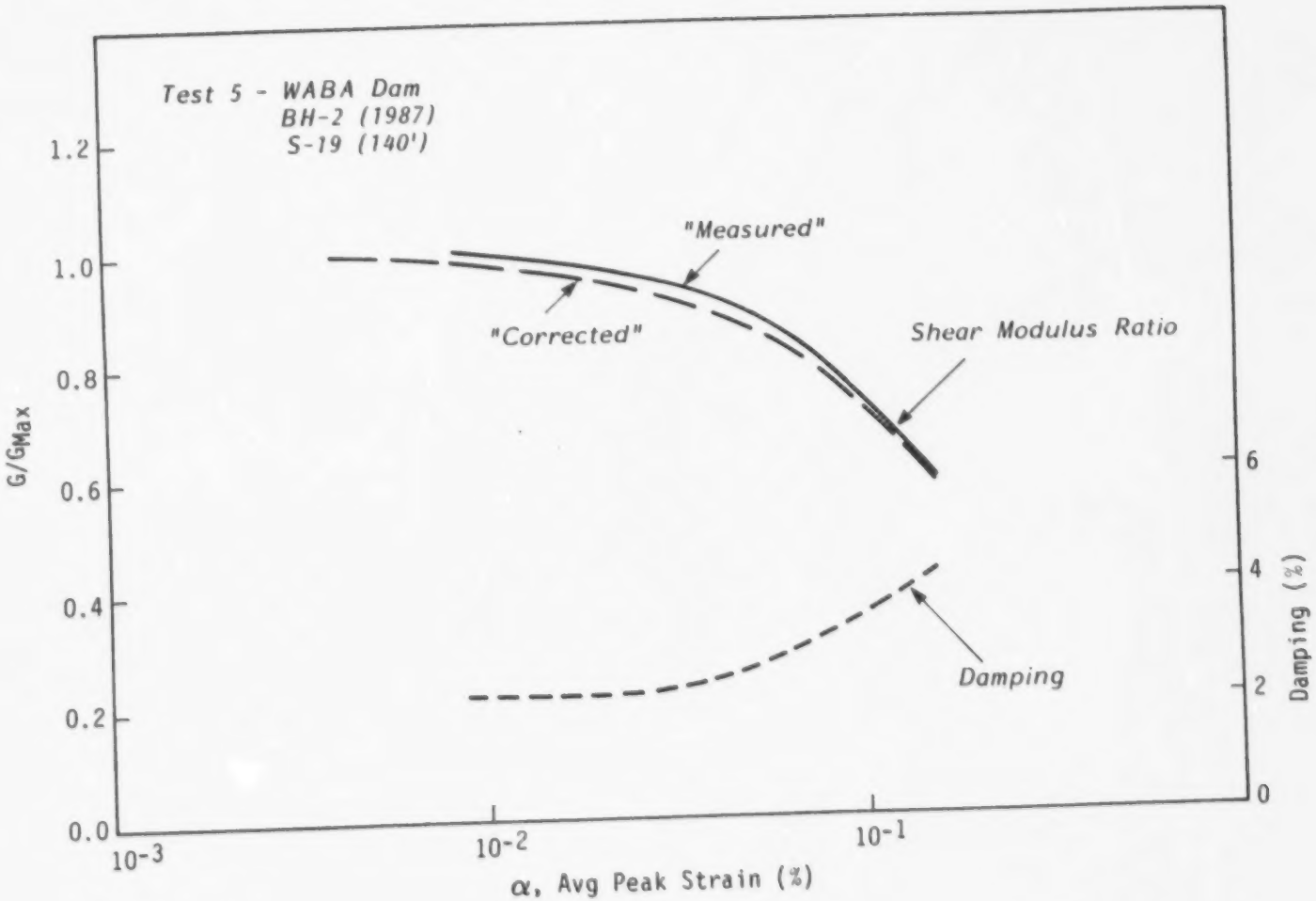

Test 5 - WABA Dam
BH-2 (1987)
S-19 (140')
"Measured"
"Corrected"
Shear Modulus Ratio
Damping
G/GMax
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
6
4
2
0
10-3
10-2
10-1

FIGURE C7

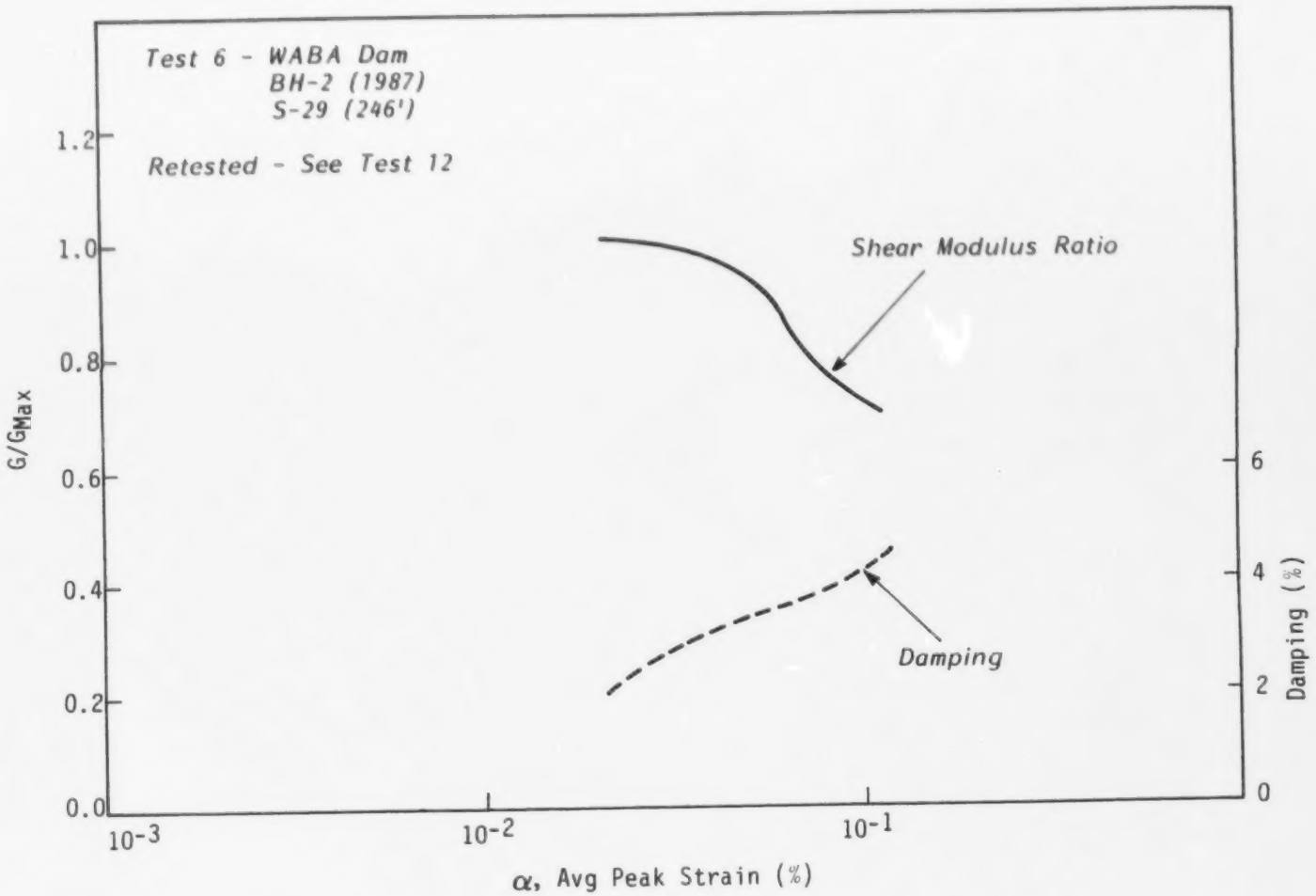

Test 6 - WABA Dam
BH-2 (1987)
S-29 (246')
Retested - See Test 12
Shear Modulus Ratio
Damping
G/GMax
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
6
4
2
0
10-3
10-2
10-1

FIGURE C8

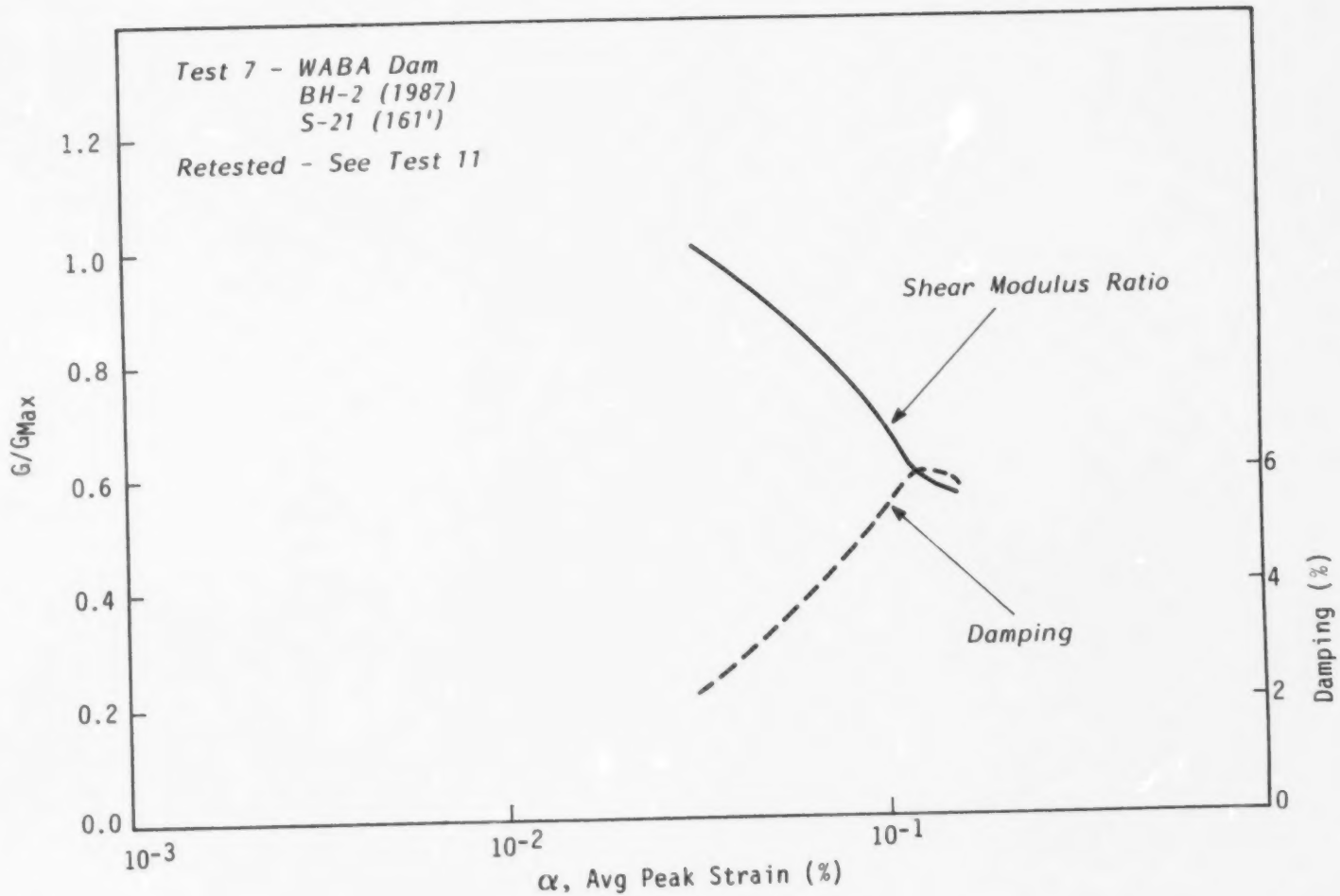

Test 7 - WABA Dam
BH-2 (1987)
S-21 (161')
Retested - See Test 11
Shear Modulus Ratio
Damping
G/GMax
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
10-3
10-2
10-1
6
4
2
0

FIGURE C9

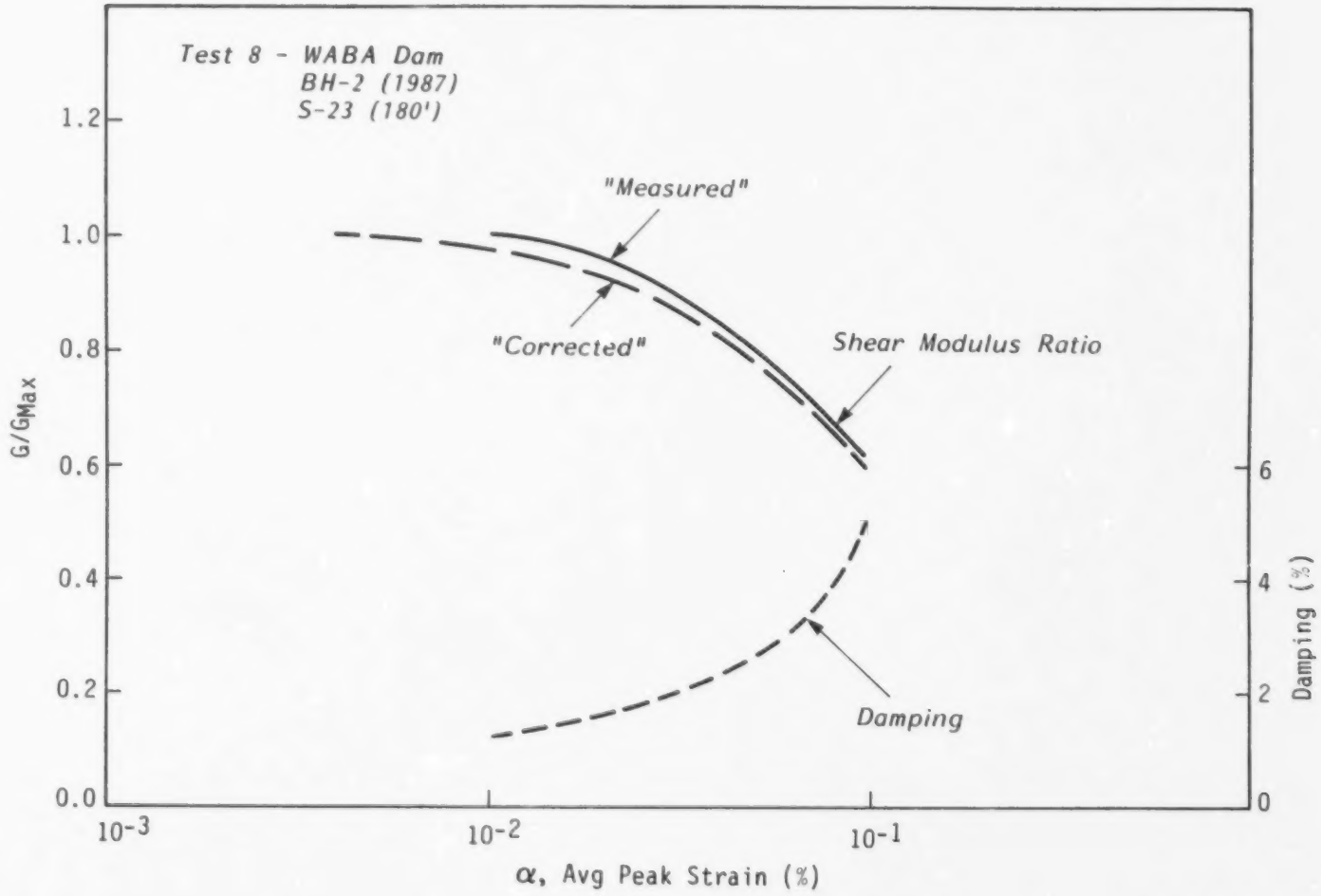

Test 8 - WABA Dam
BH-2 (1987)
S-23 (180')
"Measured"
"Corrected"
Shear Modulus Ratio
Damping
G/GMax
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
6
4
2
0
10-3
10-2
10-1

FIGURE C10

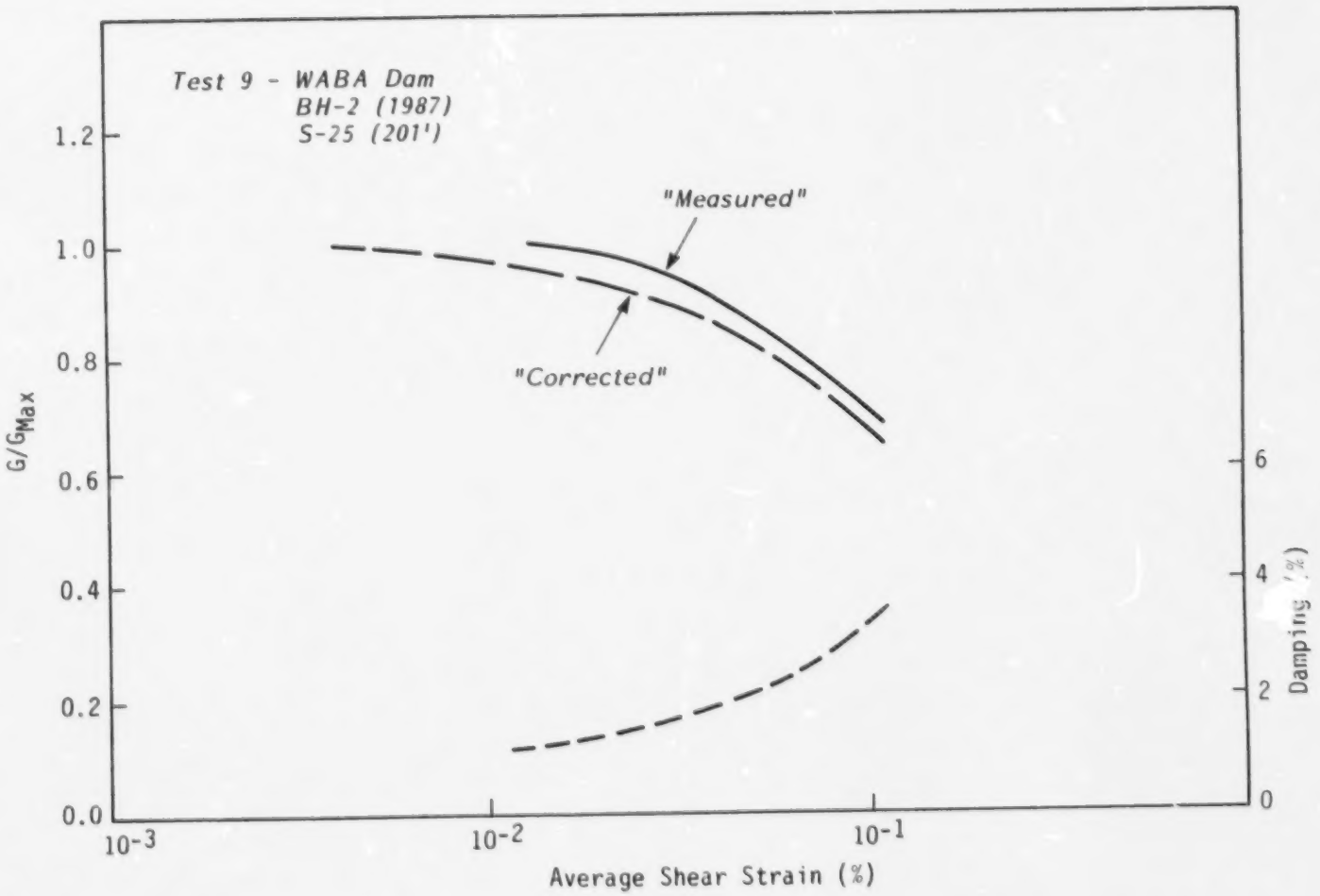

Test 9 - WABA Dam
BH-2 (1987)
S-25 (201')
"Measured"
"Corrected"
G/GMax
1.2
1.0
0.8
0.6
0.4
0.2
0.0
10-3
10-2
10-1
Average Shear Strain (%)
6
4
2
0
Damping (%)

FIGURE C11

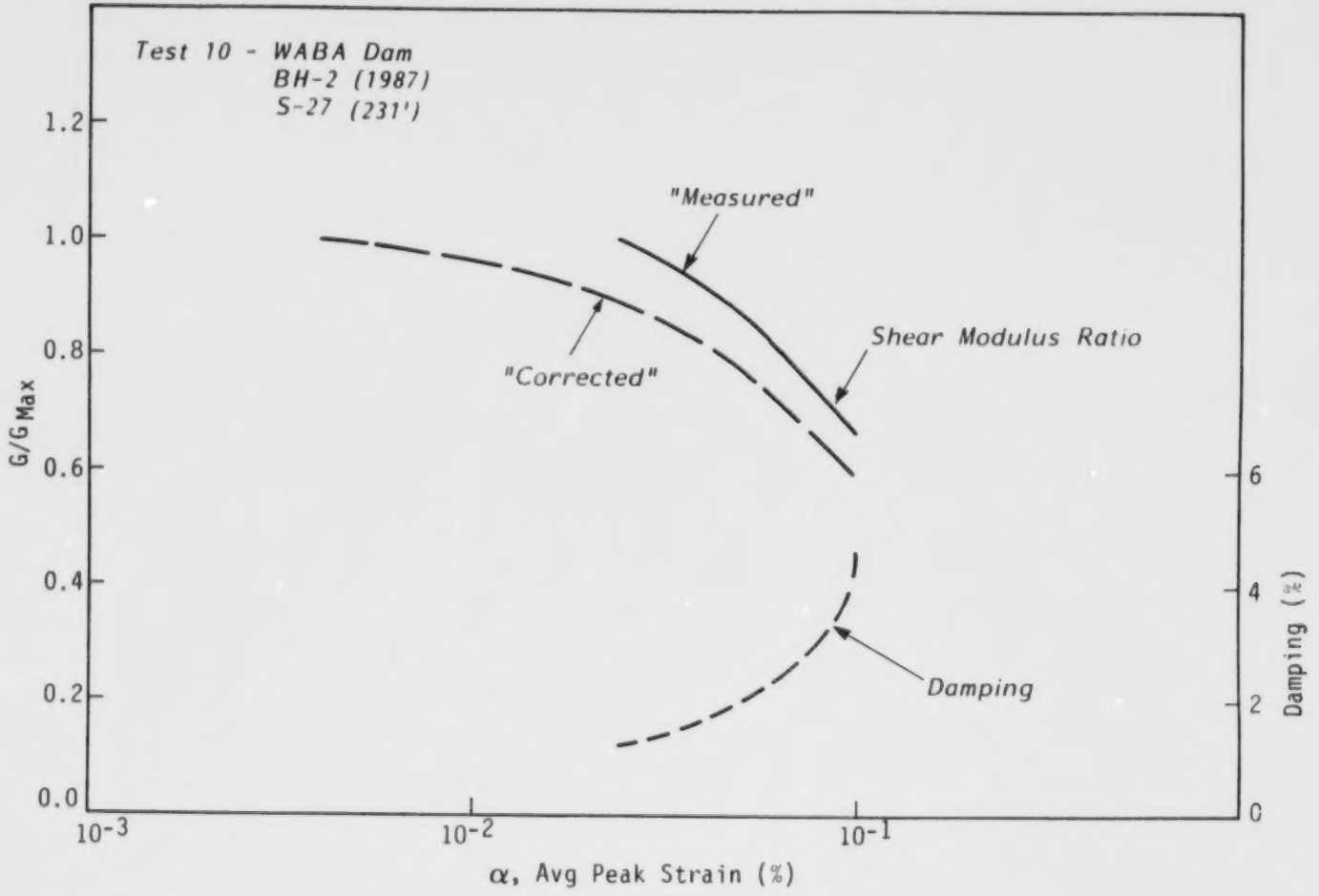

Test 10 - WABA Dam
BH-2 (1987)
S-27 (231')
"Measured"
"Corrected"
Shear Modulus Ratio
Damping
G/G Max
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
6
4
2
0
10-3
10-2
10-1

FIGURE C12

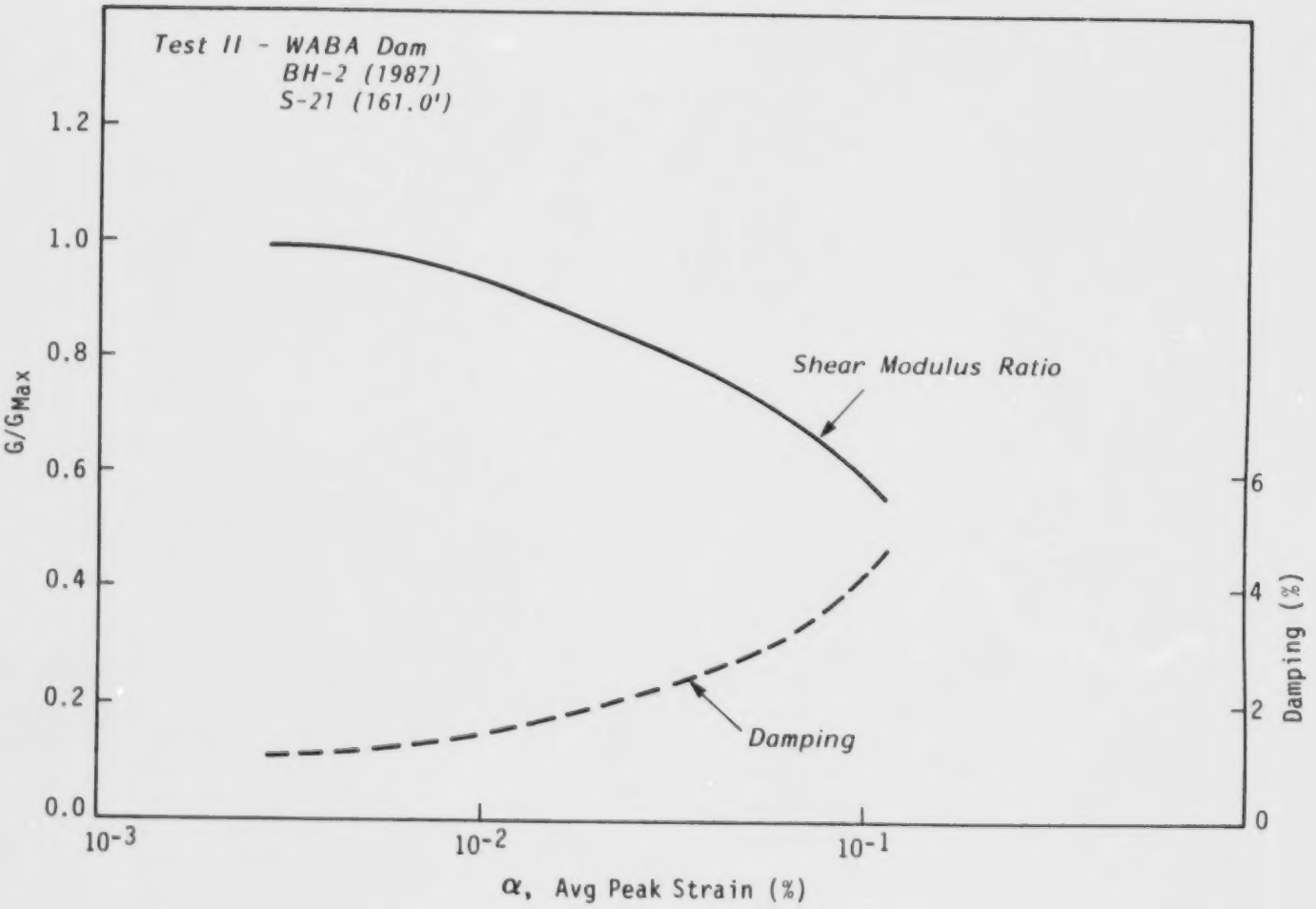

Test II - WABA Dam
BH-2 (1987)
S-21 (161.0')
Shear Modulus Ratio
Damping
G/GMax
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
6
4
2
0
10-3
10-2
10-1

FIGURE C13

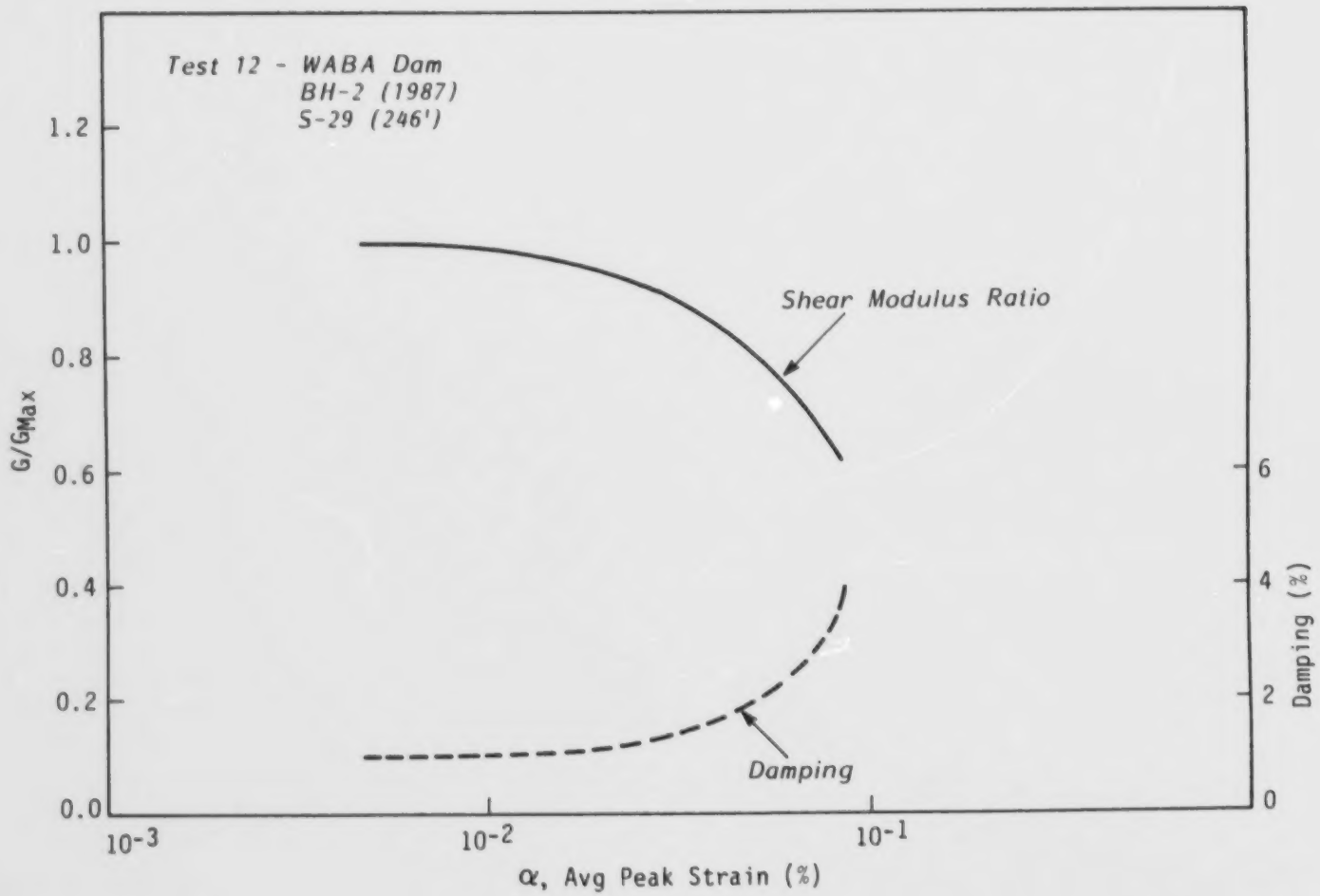

Test 12 - WABA Dam
BH-2 (1987)
S-29 (246')
Shear Modulus Ratio
Damping
G/GMax
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
6
4
2
0
10-3
10-2
10-1

FIGURE C14

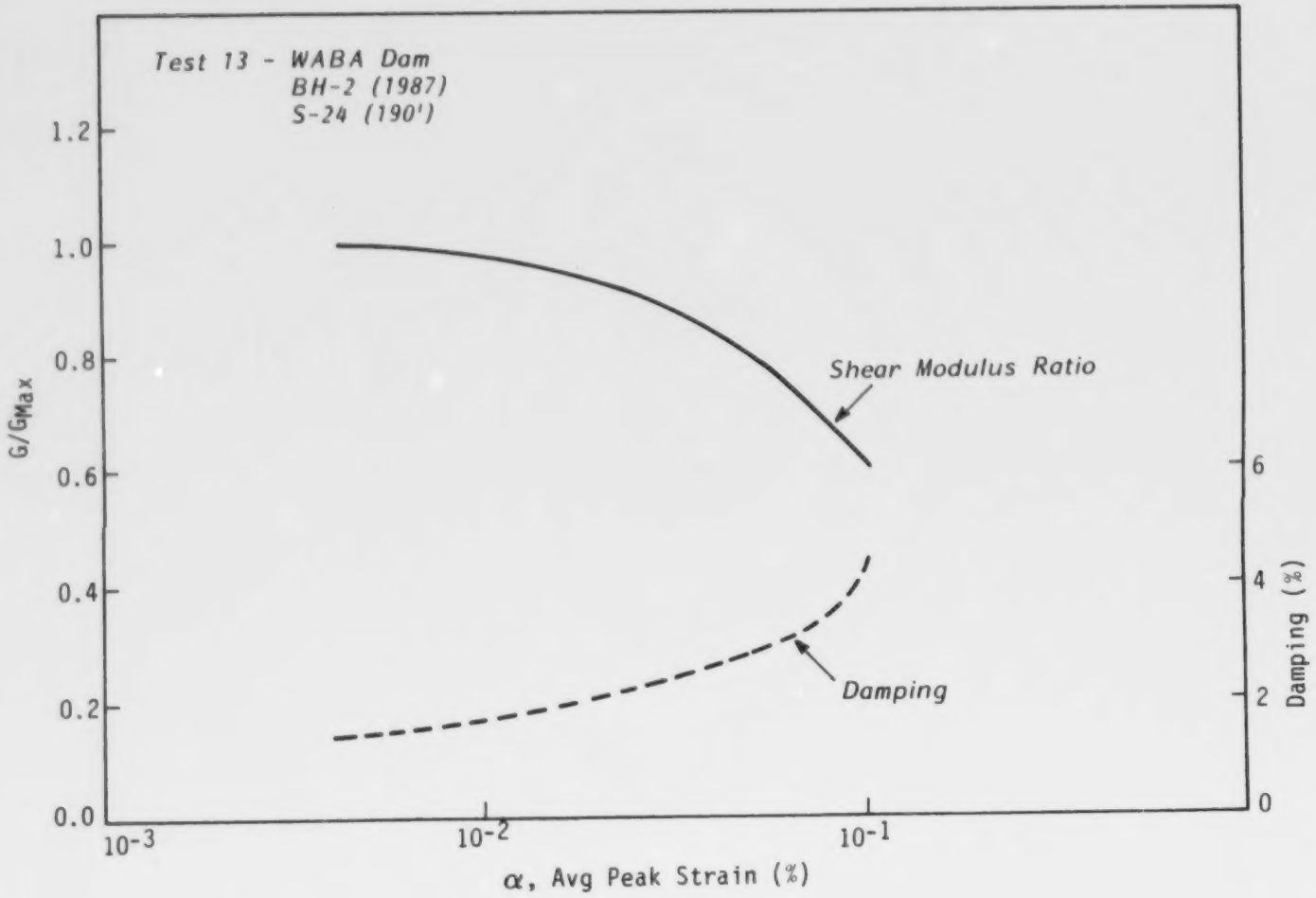

Test 13 - WABA Dam
BH-2 (1987)
S-24 (190')
Shear Modulus Ratio
Damping
G/GMax
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
6
4
2
0
10-3
10-2
10-1

FIGURE C15

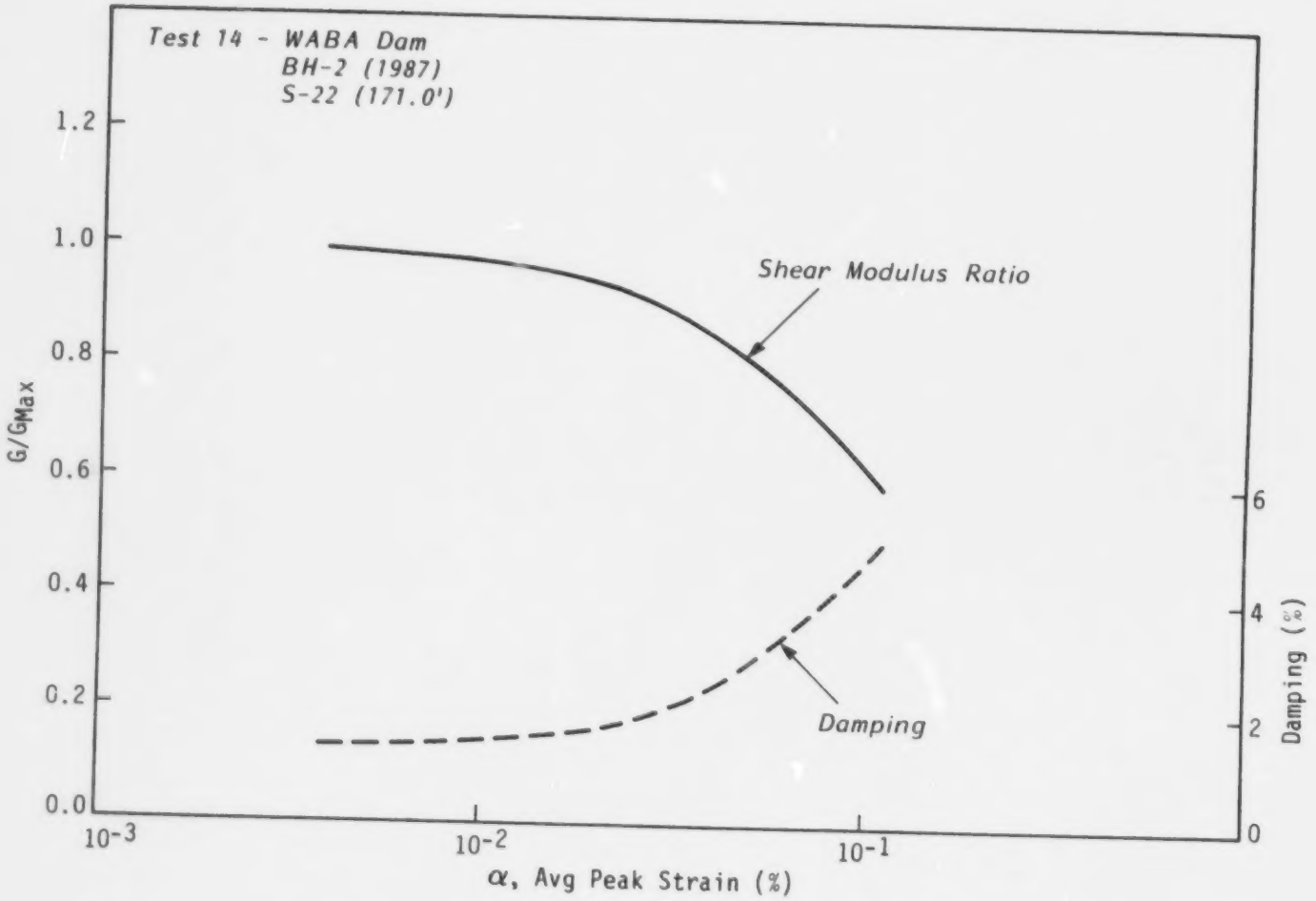

Test 14 - WABA Dam
BH-2 (1987)
S-22 (171.0')
Shear Modulus Ratio
Damping
G/GMax
Damping (%)
α, Avg Peak Strain (%)
1.2
1.0
0.8
0.6
0.4
0.2
0.0
10-3
10-2
10-1
6
4
2
0

FIGURE C16

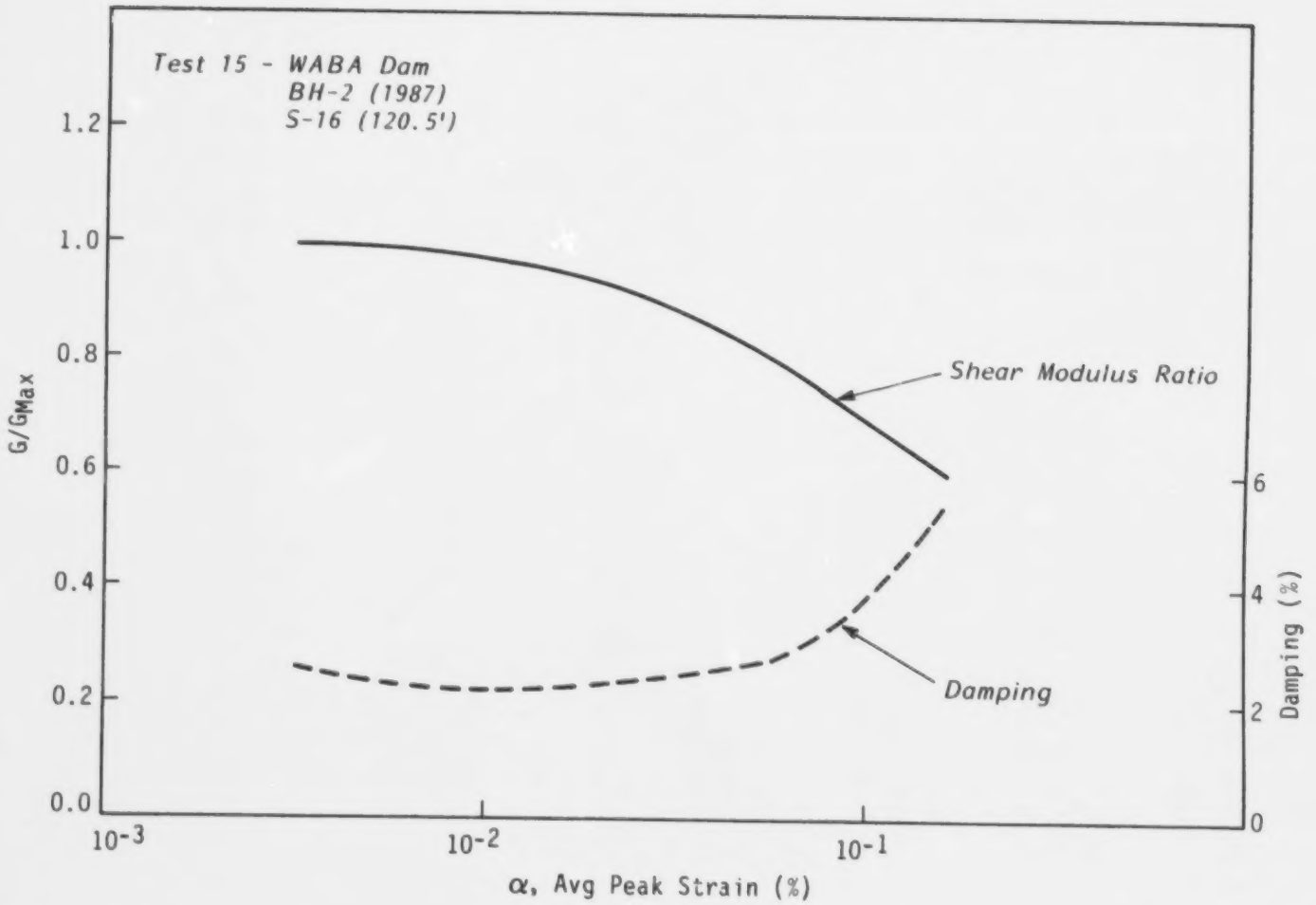

Test 15 - WABA Dam
BH-2 (1987)
S-16 (120.5')
1.2
1.0
0.8
0.6
0.4
0.2
0.0
G/GMax
Shear Modulus Ratio
Damping
6
4
2
0
Damping (%)
10-3
10-2
10-1
α, Avg Peak Strain (%)

FIGURE C17

APPENDIX D

DYNAMIC TRIAXIAL TESTS

TABLE D1

Dynamic Triaxial Test Summary

Set	Sample	Depth (ft)	Bulk Density (pcf)	Water Content (%)	Confining Pressure (psf)	Stress Ratio	Cycle to Failure	Comments
A	S-11/1	91	97.6	79.5	3130	0.28	N/A	<1% strain at 620 cycles
	S-11/1	91			3130	0.34	N/A	<1% strain at 320 cycles
	S-11/1	91			3130	0.40	N/A	<1% strain at 324 cycles
	S-11/1	91			3130	0.45	436	
B	S-5/1	63	98.2	76.4	2300	0.29	227	
	S-5/2	63	98.7	74.9	2300	0.36	68	
	S-5/3	63	99.6	71.5	2300	0.42	31	
C	S-15/1	110	100.5	64.9	3760	0.27	101	
	S-15/2	110	102.1	62.9	3760	0.35	38	
	S-15/3	110	100.3	62.6	3760	0.42	3	
D	S-25/1	201	108.4	49.0	7940	0.30	99	
	S-25/2	201	107.7	50.5	7940	0.36	47	
	S-24/1	191	113.9	40.0	7940	0.41	3	

TABLE D2

Post Dynamic Strength Tests

Sample Number	Depth (ft)	Bulk Density (pcf)	Water Content (%)	Confining Pressure (psf)	Reference Shear Strength (psf)	Dynamic Load		Measured Shear Strength (psf)	Strength Loss (%)
						Stress Ratio	No of Cycles		
S-11	92	98.3	73.6	3030	2465 (PP)	0.32	25	2400	2.6
		98.3	73.6	3130	2465	0.31	100	2560	none
S-5	63	98.8	70.2	2300	710 (LV)	0.28	25	1350	none
		96.5	75.1	2300	630 (LV)	0.29	100	1140	none
S-15	110	103.1	57.1	3760	750	0.28	10	1260	1260
		104.1	61.4	3760	850	0.28	25	1590	none
S-24	191	112.8	40.5	7940	2160 (LV)	0.28	16	2150	none

PP - pocket penetrometer
LV - lab vane

APPENDIX E

Pore Water Chemistry

Salinity Test and Analysis Method

Pore water salinity measurements for Waba Dam clay samples were performed on water squeezed from the soil specimens listed in Table E1. Total conductivity of the pore water is actually measured in milli semens per metre (mS/m) as shown in column 3 (Table E1). Using Figure E1 or Table E2 the conductivity is correlated to a solution of sodium chloride (NaCl) salt having the equivalent conductivity. The concentration of NaCl in the equivalent solution is recorded in column 4 as the equivalent salt content of the pore water. Data from columns 4 and 6 (water content) are combined to produce salt content as a per cent of soil dry weight (column 5).

TABLE E1

Waba Dam (BH.2)
Pore Water Salinity Tests

Sample No (1)	Depth (ft) (2)	Conductivity (mS/m) (3)	Salt Content (Equivalent NaCl) (g/ℓ) (4)	Salt Content (Equivalent NaCl) (% dry soil) (5)	Water Content (%) (6)
5a	25	141	0.81	0.03	41.2
7a	35	217	1.26	0.04	29.7
9a	39	569	3.51	0.21	58.9
5	61	3,015	20.94	1.78	85.0
9	81	2,569	17.48	1.36	77.9
13	101	3,431	24.16	1.70	70.3
17	131	2,950	20.49	1.20	58.4
21	161	3,000	20.83	0.79	38.1
23	181	2,308	15.59	0.60	38.7
29	246	1,108	7.10	0.31	43.8

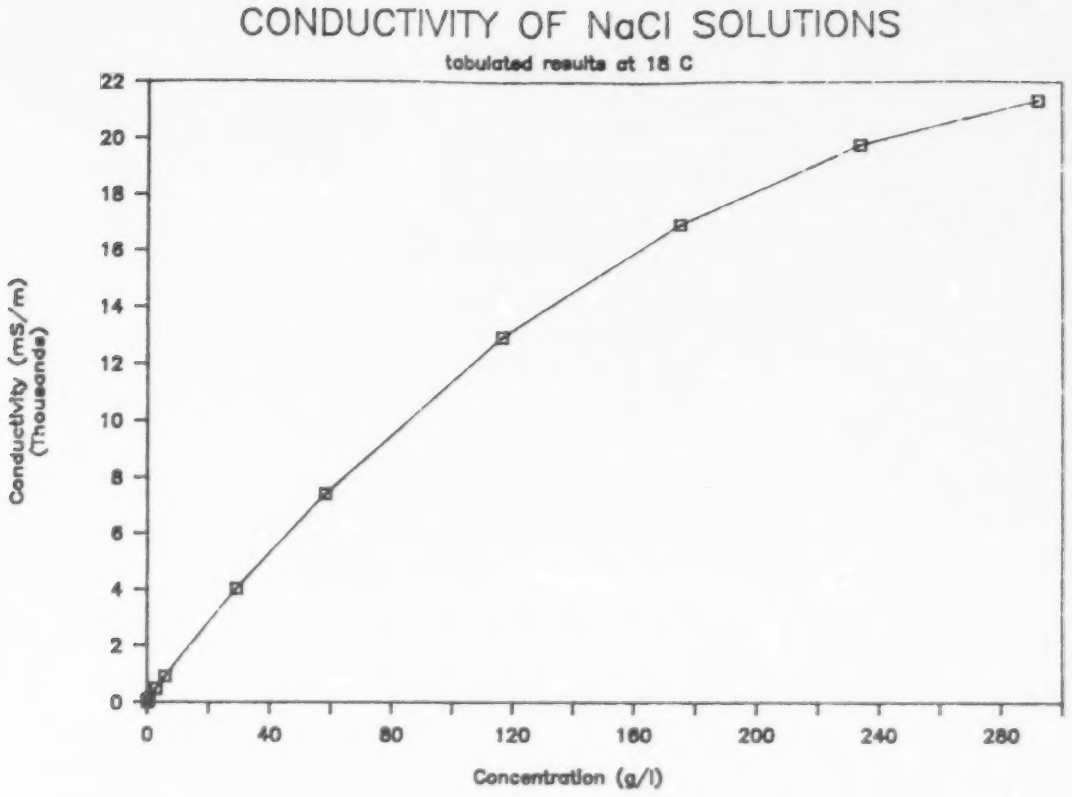
Table E2

CONDUCTIVITY/CONCENTRATION RELATIONSHIP FOR NaCl SOLUTIONS AT 18 C

Conc. (N)	Conc. (mg/l)	Conc. (g/l)	Equiv. Cond. (S*cm2*mol-1)	Cond (uS/cm)	Cond (mS/m)	Cond/ Conc
0.001	58.44	0.05844	106.5	106.5	10.65	1.82
0.005	292.2	0.2922	103.8	519	51.9	1.78
0.01	584.4	0.5844	102	1020	102	1.75
0.05	2922	2.922	95.7	4785	478.5	1.64
0.1	5844	5.844	92	9200	920	1.57
0.5	29220	29.22	80.9	40450	4045	1.38
1	58440	58.44	74.3	74300	7430	1.27
2	116880	116.88	64.8	129600	12960	1.11
3	175320	175.32	56.5	169500	16950	0.97
4	233760	233.76	49.4	197600	19760	0.85
5	292200	292.2	42.7	213500	21350	0.73

Conc. = Concentration
Cond. = Conductivity

N.B.: Concentration/Equivalent Conductivity relationship taken from Lange's Handbook of Chemistry, McGraw-Hill Book Company, 1985.



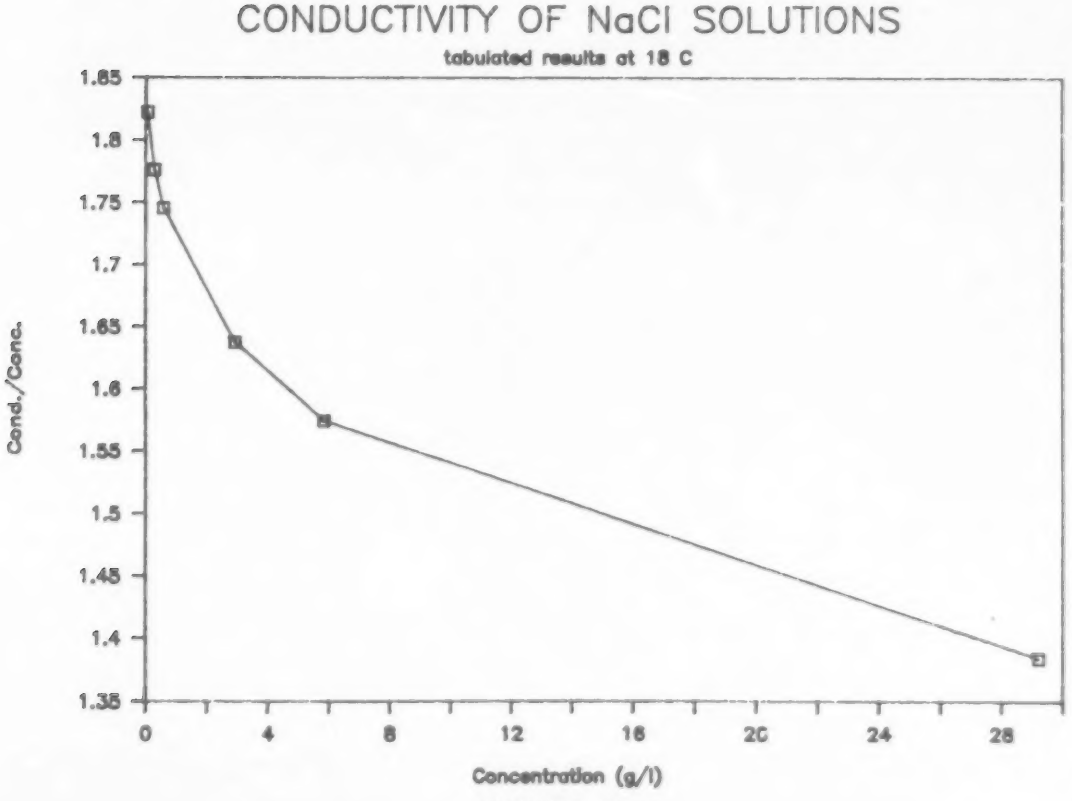


FIGURE E1

CONDUCTIVITY VERSUS CONCENTRATION OF NaCl SOLUTIONS

APPENDIX F

Swelling Tests on Waba Dam Clay Fill

TABLE F1

Swelling Test Results

Sample No	Source Borehole No	Source Depth (ft)	Bulk Density (lb/ft^3)	Water Content (%)	Settlement Due to Initial Load* (%)	Heave (Settlemen Due to Saturatior for 8 days (%)
1	Dil 1	20	122	28.4	3.4	(0.066)
2	Dil 3	10	114	34.3	2.2	(0.054)
3	Dil 4	15	124	27.5	4.5	(0.074)
4	BH 2	40	96	77.5	5.1	(0.784)

*initial load of 2800 psf applied for 24 hours before water added

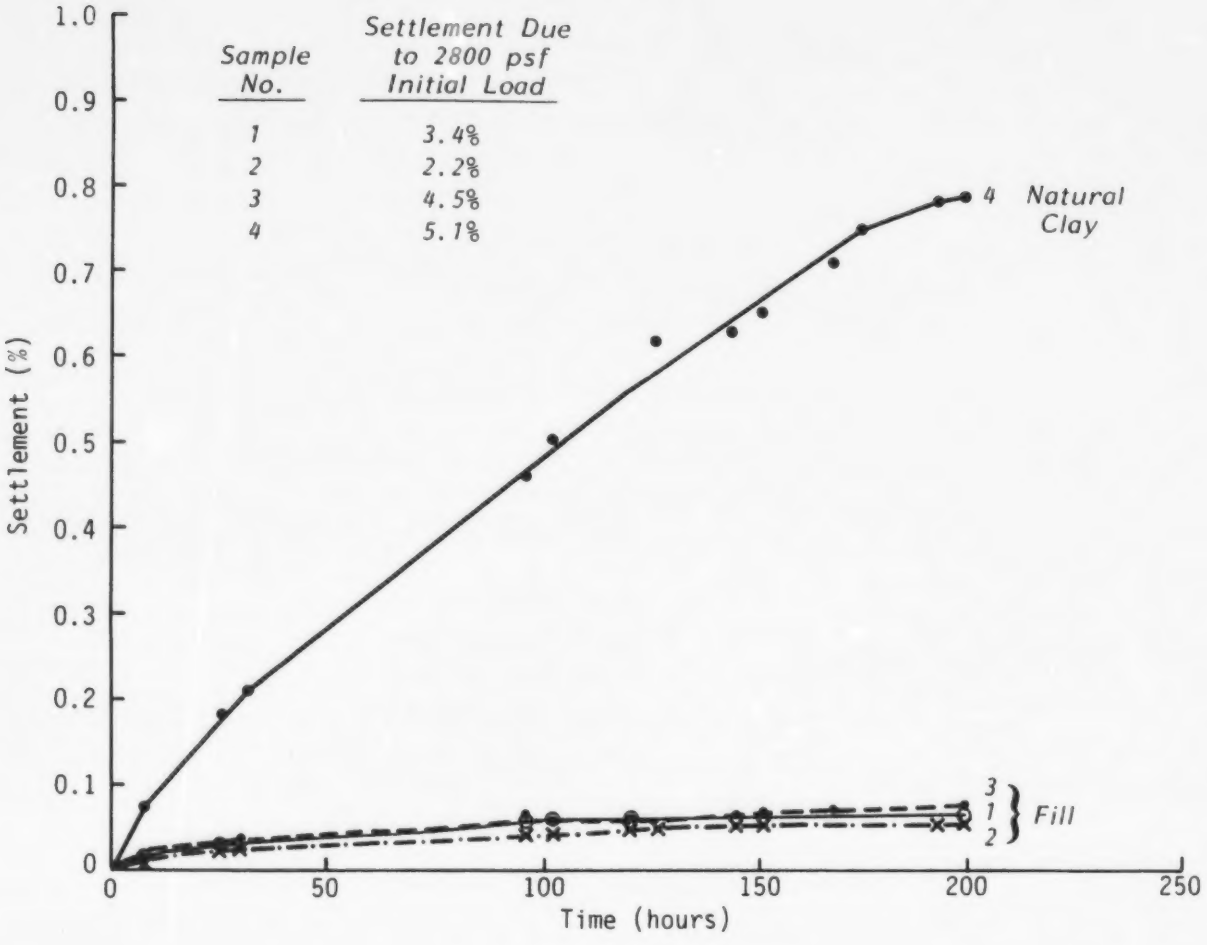


FIGURE F1

SWELLING TESTS ON WABA DAM CLAY FILL